DEPARTMENT OF THE ARMY
U.S. Army Corps of Engineers
Washington, DC 20314-1000

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Manual
No. 1110-2-1421

28 February 1999

Engineering and Design
GROUNDWATER HYDROLOGY

1. Purpose. The purpose of this manual is to provide guidance to Corps of Engineers personnel who are responsible for groundwater-related projects.

2. Applicability. This manual applies to all USACE Commands having responsibility for design of civil works projects.

3. Distribution Statement. Approved for public release, distribution is unlimited.

FOR THE COMMANDER:

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Major General, USA
Chief of Staff

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Chapter 1
Introduction

1-1. Purpose

This Engineer Manual provides guidance to Corps of Engineers (CE) personnel who are responsible for groundwater-related projects. This manual was written with special attention to groundwater-related applications prevalent within the CE. Thus, sections addressing site investigation procedures and the performance of modeling studies are included. Additionally, a chapter focusing on the interaction between surface water and groundwater is included.

1-2. Applicability

This manual applies to all USACE Commands having civil works responsibilities. This manual provides information for application to common Corps groundwater-related studies, including:

a. Site characterization for contaminant remediation.

b. Computer modeling of groundwater flow.

c. Groundwater and surface water interaction studies.

d. Reservoir operations.

e. Groundwater flow to adjacent locks and dams.

f. Remediation of reservoir leakage.

g. Infiltration of runoff to the subsurface.

h. Baseflow between aquifers and fixed bodies including streams and reservoirs.

i. Effects of aquifer pumping on adjacent lakes and streams.

j. Well installation involved with seawater infiltration barriers.

k. Dewatering of an excavation for construction purposes.

l. General regional and local applications.

1-3. References

A variety of sources were used to compile the information presented herein. This includes publications of professional societies, and guidance developed by the Corps of Engineers and other federal agencies. Appendix A contains a complete list of references. The following texts provide a general understanding of groundwater concepts and principles.

a. Driscoll, F. G. 1986. "Groundwater and Wells," 2nd ed., Johnson Wheelabrator Water Technologies, Inc., St. Paul, MN.

b. Domenico, P. A., and Schwartz, F. W. 1990. "Physical and chemical hydrogeology," John Wiley and Sons, NY.

c. Fetter, C. W. 1994. "Applied hydrogeology," 3rd ed., Charles E. Merrily Pub., Columbus, OH.

d. Freeze, R. A., and Cherry, J. A. 1979. "Groundwater," Prentice-Hall, Inc., Englewood Cliffs, NJ.

e. U.S. Department of the Interior. 1977. "Ground water manual - A water resources technical publication," U.S. Department of the Interior, Bureau of Reclamation.

f. Heath, Ralph C. 1987. "Basic ground-water hydrology," U.S. Geological Survey *Water-Supply Paper* 2220.

1-4. Distribution Statement

Approved for public release; distribution is unlimited.

1-5. Focus

This manual focuses on areas of particular concern to Corps projects. In the past 10 years, significant technical progress has been made in the field of computer modeling of groundwater flow. These new modeling technologies have had widespread applications within the Corps. This manual provides specific information regarding the performance of a site investigation and conducting a modeling study. Additionally, a significant portion of Corps applications are involved with surface water. The interrelationship of surface water and groundwater should be considered on all Corps surface water and applicable groundwater projects. This manual addresses analytical and numerical methods for quantifying the water exchange between surface water and groundwater.

1-6. Approach

This manual is intended for the use by Corps personnel in planning and designing groundwater-related projects. In many field applications, it is not possible to provide specific instructions and/or specific procedures that are universally applicable to every situation that may be encountered. Therefore, this manual emphasizes the use of sound judgement and the development of a good understanding of basic groundwater concepts rather than providing specific guidelines.

1-7. Scope

The manual provides a general overview of groundwater principles. Practical discussions are provided for planning groundwater investigations and modeling of groundwater flow. Additionally, a section on surface water and groundwater interaction is included. To enhance understanding of concepts, examples are provided throughout the document.

1-8. Format

This manual initially presents an overview of the occurrence and movement of groundwater. Procedures for planning and managing a site characterization and modeling study are then presented. This is followed by chapters addressing the technical aspects of field investigative methods and computer modeling. A final chapter discussing the interaction of groundwater and surface water is then presented. Appendices are included that contain detailed references, definitions, and additional supporting information.

a. Chapter 2. "Occurrence and Movement of Groundwater," presents an overview of general concepts. For Corps-specific applications, a section on estimating the capture zones of pumping wells is included.

b. Chapter 3. "Planning a Groundwater Investigation and Modeling Study," provides general guidelines for performing a site characterization, and integrating hydrogeologic information into a computer model. This includes: initial site reconnaissance, data interpretation, acquisition of additional data, conceptual model formulation, and general steps in developing a groundwater flow model. Additionally, project management guidelines are included.

c. Chapter 4. "Field Investigative Methods." Adequate conceptualization of a hydrogeologic system often requires the acquisition of new field data. This chapter provides an overview of different methods that can be employed to gain a better understanding of subsurface conditions. Key references are provided to allow for a more detailed understanding of concepts and applications.

d. Chapter 5. "Computer Modeling of Groundwater Flow," presents a technical overview of numerical modeling of groundwater flow.

e. Many Corps projects are related to the interaction of groundwater and surface water. Chapter 6, "Interaction Between Surface Water and Groundwater," provides an overview of the distribution and movement of water in the subsurface. Practical analytical methods which quantify the interaction between surface water and groundwater are presented. Numerical models are often employed to quantify the water exchange between the surface and subsurface. This chapter presents an overview of current technology available for the simulation of interaction between surface water and groundwater. Key references are provided to allow for a more detailed understanding of concepts and applications.

Chapter 2
Occurrence and Movement of Groundwater

2-1. General

The occurrence and movement of groundwater are related to physical forces acting in the subsurface and the geologic environment in which they occur. This chapter presents a general overview of basic concepts which explain and quantify these forces and environments as related to groundwater. For Corps-specific applications, a section on estimating capture zones of pumping wells is included. Additionally, a discussion on saltwater intrusion is included. For a more detailed understanding of general groundwater concepts, the reader is referred to Fetter (1994).

2-2. Hydrologic Cycle

a. The Earth's hydrologic cycle consists of many varied and interacting processes involving all three phases of water. A schematic diagram of the flow of water from the atmosphere, to the surface and subsurface, and eventually back to the atmosphere is shown in Figure 2-1.

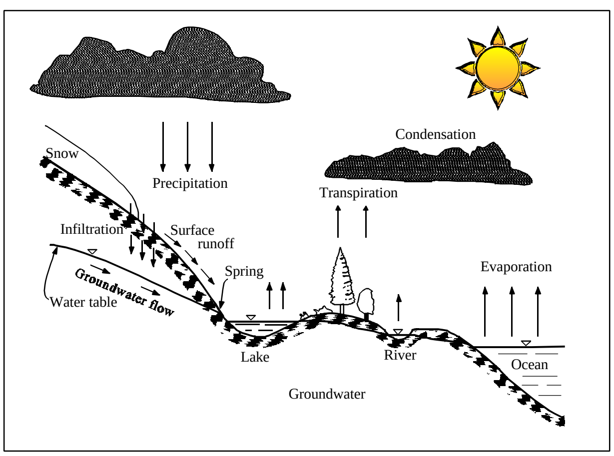

Figure 2-1. Hydrologic cycle

b. Groundwater flow is but one part of this complex dynamic hydrologic cycle. Saturated formations below the surface act as mediums for the transmission of groundwater, and as reservoirs for the storage of water. Water infiltrates to these formations from the surface and is transmitted slowly for varying distances until it returns to the surface by action of natural flow, vegetation, or man (Todd 1964). Groundwater is the largest source of available water within the United States, accounting for 97 percent of the available fresh water in the United States, and 23 percent of fresh-water usage (Solley and Pierce 1992).

2-3. Subsurface Distribution

a. General. Groundwater occurs in the subsurface in two broad zones: the unsaturated zone and the saturated zone. The unsaturated zone, also known as the vadose zone, consists of soil pores that are filled to a varying degree with air and water. The zone of saturation consists of water-filled pores that are assumed to be at hydrostatic pressure. For an unconfined aquifer, the zone of saturation is overlain by an unsaturated zone that extends from the water table to the ground surface (Figure 2-2).

b. Unsaturated zone. The unsaturated zone (or vadose zone) serves as a vast reservoir which, when recharged, typically discharges water to the saturated zone for a relatively long period after cessation of surface input. The unsaturated zone commonly consists of three sub-zones: the root zone, an intermediate zone, and the capillary fringe. The root zone varies in thickness depending upon growing season and type of vegetation. The water content in the root zone is usually less than that of saturation, except when surface fluxes are of great enough intensity to saturate the surface. This region is subject to large fluctuations in moisture content due to evaporation, plant transpiration, and precipitation. Water below the root zone is either percolating near vertically downward under the influence of gravity, or is suspended due to surface tension after gravity drainage is completed. This intermediate zone does not exist where the capillary fringe or the water table intercepts the root zone. The capillary fringe extends from the water table up to the limit of capillary rise. Water molecules at the water surface are subject to an upward attraction due to surface tension of the air-water interface and the molecular attraction of the liquid and solid phases. The thickness of this zone depends upon the pore size of the soil medium, varying directly with decrease in pore size. Water content can range from very low to saturated, with the

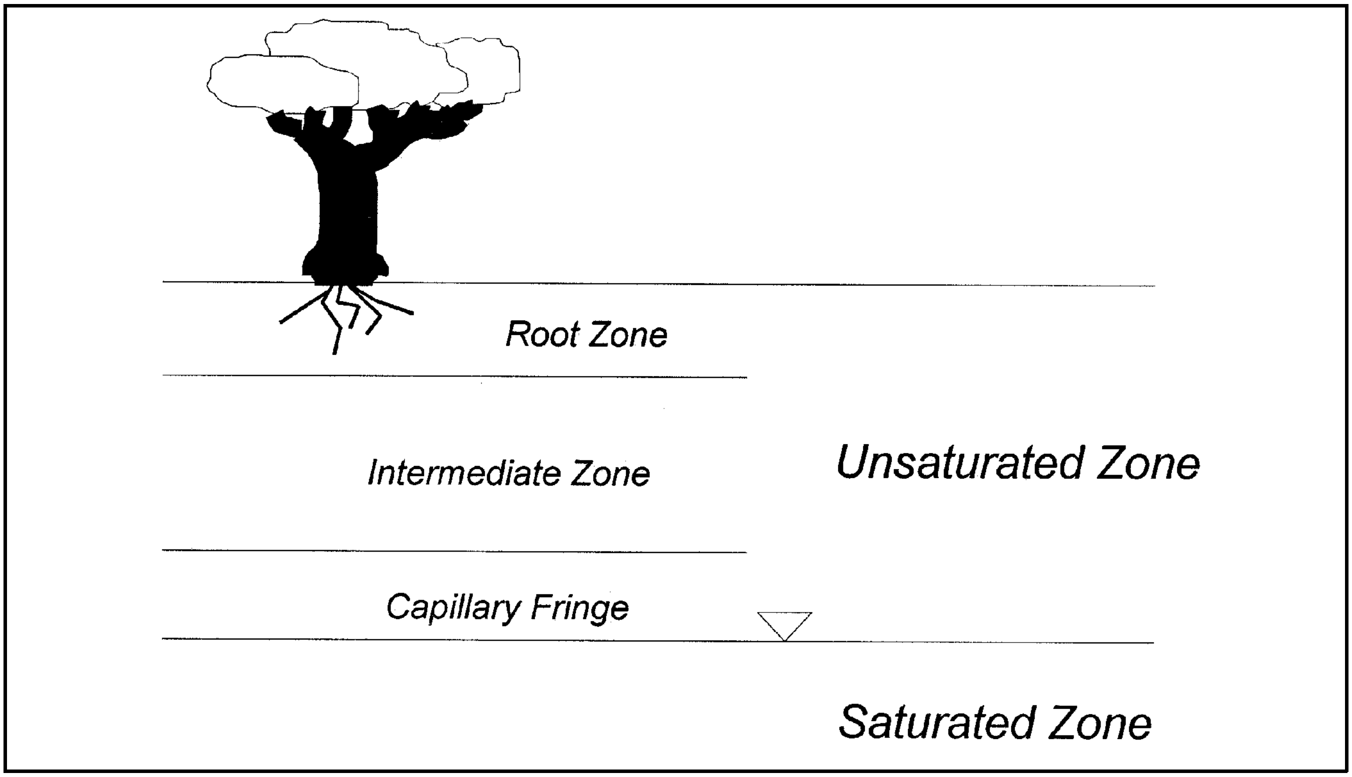

Figure 2-2. Subsurface distribution of water

lower part of the capillary fringe often being saturated. Infiltration and flow in the unsaturated zone are discussed in Section 6-3.

c. Saturated zone. In the zone of saturation, all communicating voids are filled with water under hydrostatic pressure. Water in the saturated zone is known as groundwater or phreatic water.

2-4. Forces Acting on Groundwater

External forces which act on water in the subsurface include gravity, pressure from the atmosphere and overlying water, and molecular attraction between solids and water. In the subsurface, water can occur in the following: as water vapor which moves from regions of higher pressure to lower pressure, as condensed water which is absorbed by dry soil particles, as water which is retained on particles under the molecular force of adhesion, and as water which is not subject to attractive forces towards the surface of solid particles and is under the influence of gravitational forces. In the saturated zone, groundwater flows through interconnected voids in response to the difference in fluid pressure and elevation. The driving force is measured in terms of hydraulic head. Hydraulic head (or potentiometric head) is defined by Bernoulli's equation:

$$h = z + \frac{p}{\rho g} + \frac{v^2}{2g} \tag{2-1}$$

where

h = hydraulic head

z = elevation above datum

p = fluid pressure with constant density ρ

g = acceleration due to gravity

v = fluid velocity

Pressure head (or fluid pressure) h_p is defined as:

$$h_p = \frac{p}{\rho g} \qquad (2\text{-}2)$$

By convention, pressure head is expressed in units above atmospheric pressure. In the unsaturated zone, water is held in tension and pressure head is less than atmospheric pressure ($h_p < 0$). Below the water table, in the saturated zone, pressure head is greater than atmospheric pressure ($h_p > 0$). Because groundwater velocities are usually very low, the velocity component of hydraulic head can be neglected. Thus, hydraulic head can usually be expressed as:

$$h = z + h_p \qquad (2\text{-}3)$$

Figure 2-3 depicts Equation 2-3 within a well.

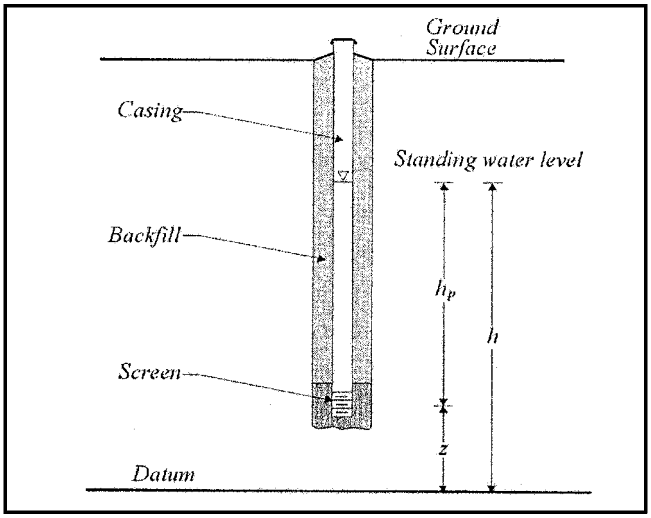

Figure 2-3. Relationship between hydraulic head, pressure head, and elevation head within a well

2-5. Water Table

As illustrated by Figure 2-3, the height of water measured in wells is the sum of elevation head and pressure head, where the pressure head is equal to the height of the water column above the screened interval within the well. Freeze and Cherry (1979) define the water table as located at the level at which water stands within a shallow well which penetrates the surficial deposits just deeply enough to encounter standing water. Thus, the hydraulic head at the water table is equal to the elevation head; and the pore water pressure at the water table is equivalent to atmospheric pressure.

2-6. Potentiometric Surface

The water table is defined as the surface in a groundwater body at which the pressure is atmospheric, and is measured by the level at which water stands in wells that penetrate the water body just far enough to hold standing water. The potentiometric surface approximates the level to which water will rise in a tightly cased well which can be screened at the water table or at greater depth. In wells that penetrate to greater depths within the aquifer, the potentiometric surface may be above or below the water table depending on whether an upward or downward component of flow exists. The potentiometric surface can vary with the depth of a well. In confined aquifers (Section 2-6), the potentiometric surface will rise above the aquifer surface. The water table is the potentiometric surface for an unconfined aquifer (Section 2-6). Where the head varies appreciably with depth in an aquifer, a potentiometric surface is meaningful only if it describes the static head along a particular specified stratum in that aquifer. The concept of potentiometric surface is only rigorously valid for defining horizontal flow directions from horizontal aquifers.

2-7. Aquifer Formations

a. General. An aquifer is a geologic unit that can store and transmit water. Aquifers are generally categorized into four basic formation types depending on the geologic environment in which they occur: unconfined, confined, semi-confined, and perched. Figure 2-4 describes basic aquifer formations.

b. Unconfined aquifers. Unconfined aquifers contain a phreatic surface (water table) as an upper boundary that fluctuates in response to recharge and discharge (such as from a pumping well). Unconfined aquifers are generally close to the land surface, with continuous layers of materials of high intrinsic permeability (Section 2-11) extending from the land surface to the base of the aquifer.

c. Confined aquifers. Confined, or artesian, aquifers are created when groundwater is trapped between

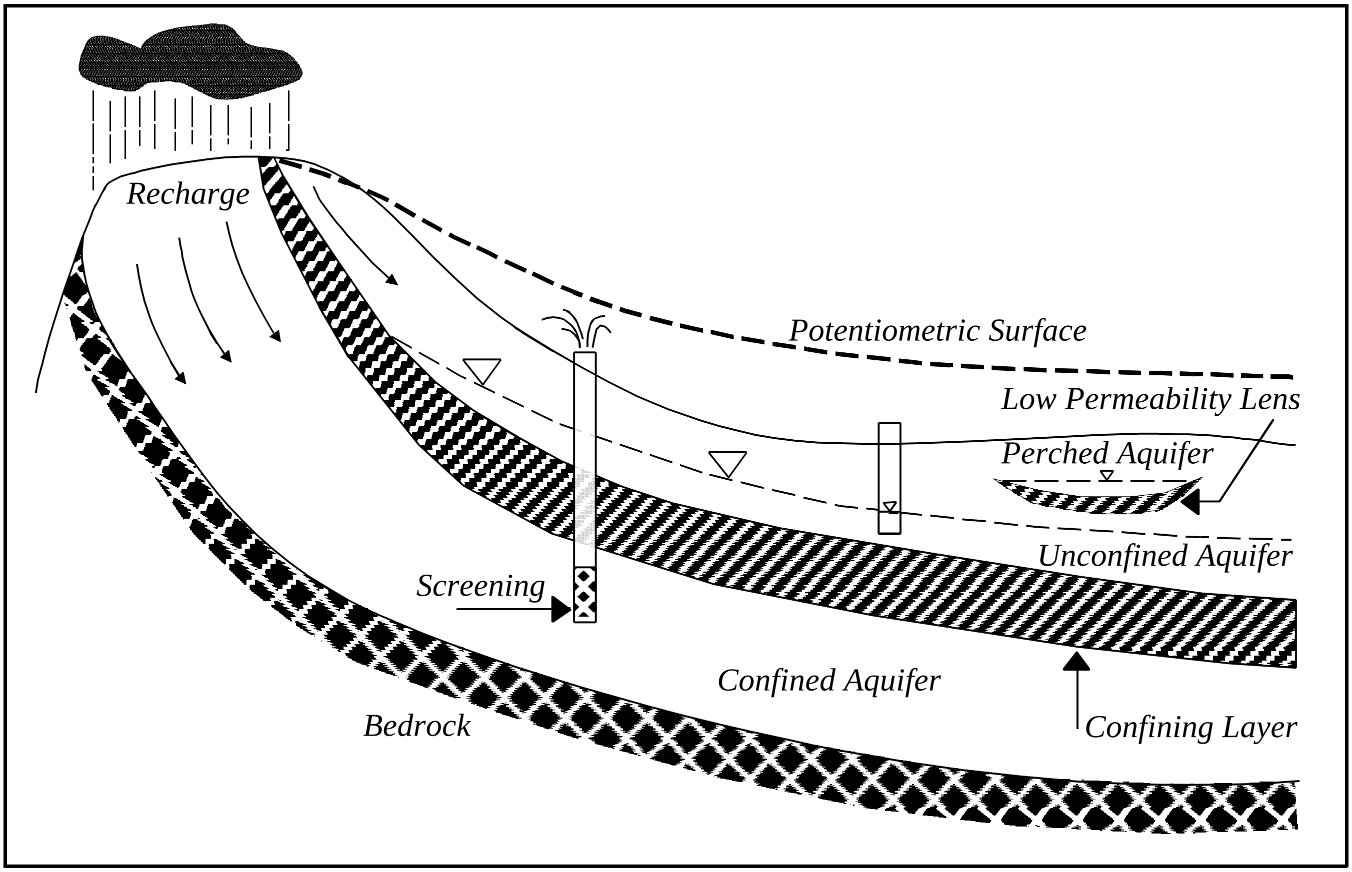

Figure 2-4. Aquifer formations

two layers of low permeability known as aquitards. In a confined aquifer, the groundwater is under pressure and the water level in a well rises above the upper boundary of the aquifer. Flowing artesian conditions exist when the water level in a well rises above land surface. Recharge to confined aquifers is predominantly from areas where the confining bed is breached, either by erosional unconformity, fracturing, or depositional absence.

d. Semi-confined aquifers. Semi-confined, or leaky, aquifers occur when water-bearing strata are confined, either above or below, by a semipermeable layer. When water is pumped from a leaky aquifer, water moves both horizontally within the aquifer and vertically through the semipermeable layer.

e. Perched aquifers. A perched aquifer is a special type of unconfined aquifer where a groundwater body is separated above the water table by a layer of unsaturated material. A perched aquifer occurs when water moving down through the unsaturated zone is intercepted by an impermeable formation. Clay lenses in sedimentary deposits often have shallow perched water bodies overlying them. Wells tapping perched aquifers generally yield temporary or small quantities of water.

2-8. Principal Types of Aquifer Materials

a. General. Earth materials which can have the potential to transmit water can be classified into four broad groups: unconsolidated materials, porous sedimentary rock, porous volcanics, and fractured rock. In unconsolidated material, water is transported through the primary openings in the rock/soil matrix. Consolidation is the process where loose materials become firm and coherent. Sandstone and conglomerate are common consolidated sedimentary rocks formed by compaction and cementation. Carbonate rocks (such as limestone and dolomite) are sedimentary rocks which can be formed by chemical precipitation. Water is usually transported through secondary openings in carbonate rocks enlarged by the dissolution of rock by water. The movement of water through volcanics and fractured rock is dependent upon the interconnectedness and frequency of flow pathways.

b. Gravel and sand. Gravel and sand aquifers are the source of most water pumped in the United States. Gravels and sands originate from alluvial, lacustrine, marine, or eolian glacial deposition.

(1) Alluvial deposits. Alluvial deposits of perennial streams are usually fairly well sorted and therefore permeable. Ephemeral streams typically deposit sand and gravel with much less sorting. Stream channels are sensitive to changes in sediment load, gradient, and velocity. This can result in lateral distribution of alluvial deposits over large areas. Areas with greater streambed slope typically contain coarser deposits. Alluvial fans occur in arid or semiarid regions where a stream issues from a narrow canyon onto a plain or valley floor. Viewed from above, they have the shape of an open fan, the apex being at the valley mouth. These alluvial deposits are coarsest at the point where the stream exits the canyon mouth, and become finer with increasing distance from the point of initial deposition.

(2) Lacustrine deposits. The central and lower portions of alluvium-filled valleys may consist of fine-grained lacustrine (or lakebed) deposits. When a stream flows into a lake, the current is abruptly checked. The coarser sediment settles rapidly to the bottom, while finer materials are transported further into the body of relatively still water. Thus, the central areas of valleys which have received lacustrine deposits often consist of finer-grained, lower-permeability materials. Lacustrine deposits usually consist of fine-grained materials that are not normally considered aquifers.

(3) Marine deposits. Marine deposits originate from sediment transported to the ocean by rivers and erosion of the ocean floor. As a sea moves inland, deposits at a point in the ocean bottom near the shore become gradually finer due to uniform wave energy. Conversely, as the sea regresses, deposition progresses gradually from finer to coarser deposits. This is a common sequence in the southern United States. Additionally, coral reefs, shells, and other calcite-rich deposits commonly occur in areas with temperate climatic conditions.

(4) Eolian deposits. Materials which are transported by the wind are known as eolian deposits. The sorting action of the wind tends to produce deposits that are uniform on a local scale, and in some cases quite uniform over large areas. Eolian deposits consist of silt or sand. Eolian sands occur wherever surface sediments are available for transport. In comparison with alluvial deposits, eolian sands are quite homogeneous and are as isotropic (Section 2-13) as any deposits occurring in nature. Eolian deposits of silt, called loess, are associated with the abnormally high wind velocities associated with glacial ice fronts. Loess occurs in the shallow subsurface in large areas of the Midwest and Great Plains regions of North America.

(5) Glacial deposits. Unlike water and wind, glacial ice can entrain unconsolidated deposits of all sizes from sediments to boulders. Glacial till is non-sorted, non-stratified sediment deposited beneath, from within, or from the top of glacial ice. Glacial outwash (or glaciofluvial) deposits consist of coarse-grained sediments deposited by meltwater in front of a glacier. The sorting and homogeneity of glaciofluvial deposits depend upon environmental conditions and distance from the glacial front.

c. Sandstone and conglomerate. Sandstone and conglomerates are the consolidated equivalents of sand and gravel. Consolidation results from compaction and cementation. The highest yielding sandstone aquifers occur where partial consolidation takes place. These yield water from the pores between grains, although secondary openings such as fractures and joints can also serve as channels of flow.

d. Carbonate rocks. Carbonate rocks, formed from calcium, magnesium, or iron, are widespread throughout the United States. Limestone and dolomite, which originate from calcium-rich deposits, are the most common carbonate rocks. Carbonates are typically brittle and susceptible to fracturing. Fractures and joints in limestone yield water in small to moderate amounts. However, because water acts as a weak acid to carbonates, dissolution of rock by water enlarges openings. The limestones that yield the highest amount of water are those in which a sizable portion of the original rock has been dissolved or removed. These areas are commonly referred to as karst. Thus, large amounts of flow can potentially be transmitted in carbonate rocks.

e. Volcanics. Basalt is an important aquifer material in parts of the western United States, most notably central Idaho, where enormous flows of lava have spread out over large areas in successive sheets of varying thickness. The ability of basalt formations to transmit water is dependent on the presence of fractures, cracks, and tubes or caverns, and can be significant. Near the surface, rapid cooling produces jointing. Fracturing below the surface occurs as the crust cools, causing differential flow velocities with depth. Other volcanic rocks, including rhyolite and other more siliceous rocks, do not usually yield water in quantities comparable to those secured from basalt. Another major source of groundwater in some parts of the western United States is found in sedimentary "interbed" materials which occur between basalt flows. This interbedded material is generally alluvial or colluvial in nature, consisting of sands, gravels, and residuum (particularly granite). When the interbedded materials tend to be finer-grained, the interbed acts as a confining layer.

f. Fractured rock. Crystalline and metamorphic rocks, including granite, basic igneous rocks, gneiss, schist, quartzite, and slate are relatively impermeable. Water in these areas is supplied as a result of jointing and fracturing. The yield of water from fractured rock is dependent upon the frequency and interconnectedness of flow pathways.

2-9. Movement of Groundwater

Groundwater moves through the sub-surface from areas of greater hydraulic head to areas of lower hydraulic head (Equation 2-3). The rate of groundwater movement depends upon the slope of the hydraulic head (hydraulic gradient), and intrinsic aquifer and fluid properties.

2-10. Porosity and Specific Yield

a. Porosity. Porosity n is defined as the ratio of void space to the total volume of media:

$$n = \frac{V_v}{V_T} \tag{2-4}$$

where

V_v = volume of void space [L^3]

V_T = total volume (volume of solids plus volume of voids) [L^3]

In unconsolidated materials, porosity is principally governed by three properties of the media: grain packing, grain shape, and grain size distribution. The effect of packing may be observed in two-dimensional models comprised of spherical, uniform-sized balls. Arranging the balls in a cubic configuration (each ball touching four other balls) yields a porosity of 0.476 whereas rhombohedral packing of the balls (each ball touching eight other balls) results in a porosity of 0.260. Porosity is not a function of grain size, but rather grain size distribution. Spherical models comprised of different sized balls will always yield a lower porosity than the uniform model arranged in a similar packing arrangement. *Primary porosity* in a material is due to the properties of the soil or rock matrix, while *secondary porosity* is developed in the material after its emplacement through such processes as solution and fracturing. Representative porosity ranges for sedimentary materials are given in Table 2-1.

Table 2-1
Porosity Ranges for Sedimentary Materials

Material	Porosity
Clay	.45 - .55
Silt	.40 - .50
Medium to coarse mixed sand	.35 - .40
Uniform sand	.30 - .40
Fine to medium mixed sand	.30 - .35
Gravel	.30 - .40
Gravel and sand	.20 - .35

b. Effective porosity. Effective porosity n_e is the porosity available for fluid flow. The effective porosity of a unit of media is equal to the ratio of the volume of interconnected pores that are large enough to contain water molecules to the total volume of the rock or soil.

c. Specific yield. Specific yield S_y is the ratio of the water that will drain from a saturated rock owing to the force of gravity to the total volume of the media. *Specific retention Sr* is defined as the ratio of the volume of water that a unit of media can retain against the attraction of gravity to the total volume of the media. The porosity of a rock is equal to the sum of the specific yield and specific retention of the media. For most practical applications in sands and gravels, the value of effective porosity can be considered equivalent to specific yield. In clays, there is a much greater surface area and corresponding adhesion of water molecules. Figure 2-5 illustrates a typical relationship of specific yield and specific retention to total porosity for different soil types.

2-11. Darcy's Law and Hydraulic Conductivity

a. Darcy's Law. Henry Darcy, a French hydraulic engineer, observed that the rate of laminar flow of a fluid (of constant density and temperature) between two points in a porous medium is proportional to the hydraulic gradient (dh/dl) between the two points (Darcy 1856). The equation describing the rate of flow through a porous medium is known as Darcy's Law and is given as:

$$Q = -KA\frac{dh}{dl} \tag{2-5}$$

where

Q = volumetric flow rate [L^3T^{-1}]

K = hydraulic conductivity [LT^{-1}]

A = cross-sectional area of flow [L^2]

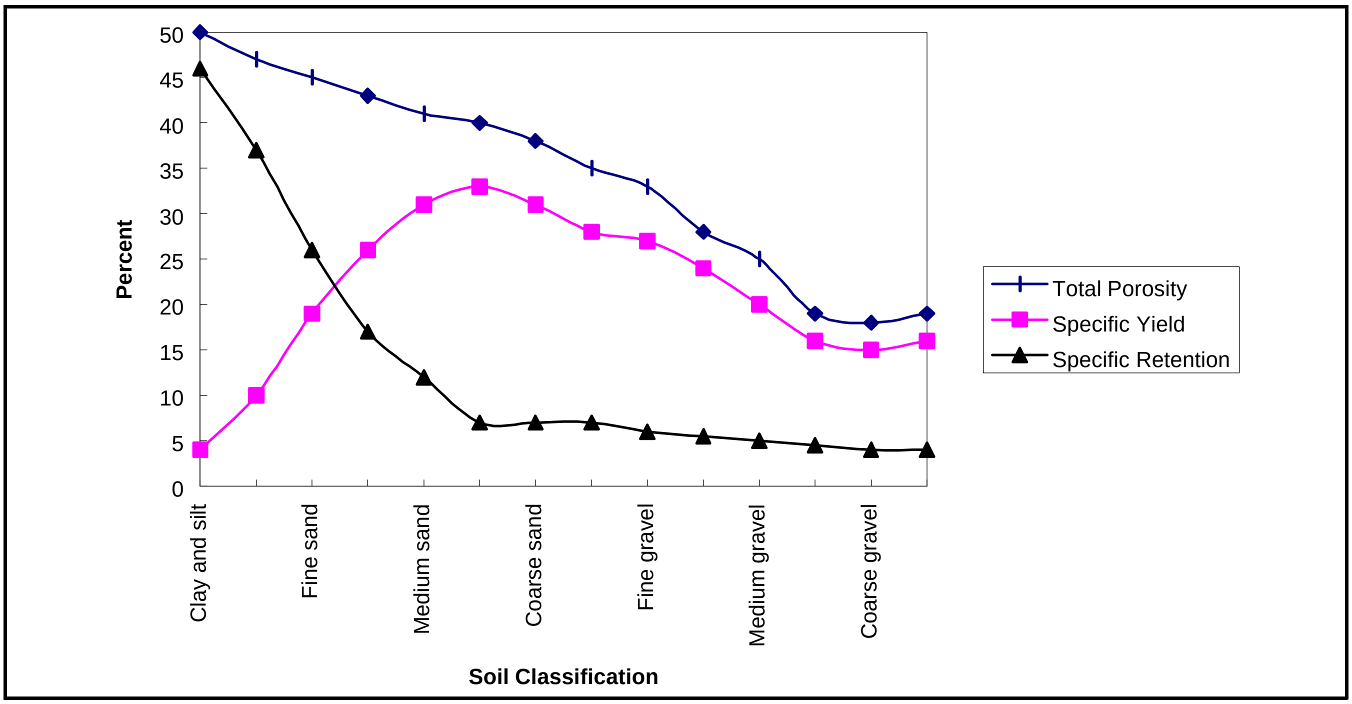

Figure 2-5. Typical relationship between specific yield, specific retention, and total porosity for different soil types

h = hydraulic head [L]

l = distance between two points [L]

The negative sign on the right-hand side of Equation 2-5 (Darcy's Law) is used by convention to indicate a downward trending flow gradient.

b. Hydraulic conductivity. The hydraulic conductivity of a given medium is a function of the properties of the medium and the properties of the fluid. Using empirically derived proportionality relationships and dimensional analysis, the hydraulic conductivity of a given medium transmitting a given fluid is given as:

$$K = \frac{k \rho g}{\mu} \tag{2-6}$$

where

k = intrinsic permeability of porous medium [L^2]

ρ = fluid density [ML^{-3}]

μ = dynamic viscosity of fluid [$ML^{-1}T^{-1}$]

g = acceleration of gravity [LT^{-2}]

The intrinsic permeability of a medium is a function of the shape and diameter of the pore spaces. Several empirical relationships describing intrinsic permeability have been presented. Fair and Hatch (1933) used a packing factor, shape factor, and the geometric mean of the grain size to estimate intrinsic permeability. Krumbein (1943) uses the square of the average grain diameter to approximate the intrinsic permeability of a porous medium. Values of fluid density and dynamic viscosity are dependent upon water temperature. Fluid density is additionally dependent upon total dissolved solids (TDS). Ranges of intrinsic permeability and hydraulic conductivity values for unconsolidated sediments are presented in Table 2-2.

c. Specific discharge. The volumetric flow velocity v can be determined by dividing the volumetric flow rate by the cross-sectional area of flow as:

Table 2-2
Ranges of Intrinsic Permeability and Hydraulic Conductivity for Unconsolidated Sediments

Material	Intrinsic Permeability (cm^2)	Hydraulic Conductivity (cm/s)
Clay	10^{-6} - 10^{-3}	10^{-9} - 10^{-6}
Silt, sandy silts, clayey sands, till	10^{-3} - 10^{-1}	10^{-6} - 10^{-4}
Silty sands, fine sands	10^{-2} - 1	10^{-5} - 10^{-3}
Well-sorted sands, glacial outwash	1 - 10^{2}	10^{-3} - 10^{-1}
Well-sorted gravels	10 - 10^{3}	10^{-2} - 1

$$v = \frac{Q}{A} = -K\frac{dh}{dl} \tag{2-7}$$

The velocity given by Equation 2-7 is termed the *specific discharge*, or *Darcy flux*. The specific discharge is actually an apparent velocity, representing the velocity at which water would move through an aquifer if the aquifer were an open conduit. The cross-sectional area is not entirely available for flow due to the presence of the porous matrix.

d. Pore water velocity. The average linear velocity of water in a porous medium is derived by dividing specific discharge by effective porosity (n_e) to account for the actual open space available for the flow. The resulting velocity is termed the *pore water velocity*, or the *seepage velocity*. The pore water velocity V_x represents the average rate at which the water moves between two points and is given by

$$V_x = \frac{Q}{n_e A} = -\frac{K dh}{n_e dl} \tag{2-8}$$

2-12. Flow and Transmissivity

Transmissivity T is a measure of the amount of water that can be transmitted horizontally through a unit width by the fully saturated thickness of an aquifer under a hydraulic gradient equal to 1. Transmissivity is equal to the hydraulic conductivity multiplied by the saturated thickness of the aquifer and is given by:

$$T = Kb \tag{2-9}$$

where

K = hydraulic conductivity [LT^{-1}]

b = saturated thickness of the aquifer [L]

Since transmissivity depends on hydraulic conductivity and saturated thickness, its value will differ at different locations within aquifers comprised of heterogeneous material, bounded by sloping confining beds, or under unconfined conditions where the saturated thickness will vary with the water table.

2-13. Homogeneity and Isotropy

a. Definition. If hydraulic conductivity is consistent throughout a formation, regardless of position, the formation is homogeneous. If hydraulic conductivity within a formation is dependent on location, the formation is heterogeneous. When hydraulic conductivity is independent of the direction of measurement at a point within a formation, the formation is isotropic at that point. If the hydraulic conductivity varies with the direction of measurement at a point within a formation, the formation is anisotropic at that point. Figure 2-6 is a graphical representation of homogeneity and isotropy.

b. Geologic controls. Geologic material is very rarely homogeneous in all directions. A more probable condition is that the properties, such as hydraulic conductivity, are approximately constant in one direction. This condition results because: a) of effects of the shape of soil particles, and b) different materials incorporate the alluvium at different locations. As geologic strata are formed, individual particles usually rest with their flat sides down in a process called imbrication. Consequently, flow is generally less restricted in the horizontal direction than the vertical and K_x is greater than K_z for most situations. Layered heterogeneity occurs when stratum of homogeneous, isotropic materials are overlain upon each other. Layered conditions commonly occur in alluvial, lacustrine, and marine deposits. At a large scale, there is a relationship between anisotropy and layered heterogeneity. In the field it is not uncommon for sites with layered heterogeneity to have large scale anisotropy values of 100:1 or greater. Discontinuous heterogeneity results from geologic structures such as bedrock outcrop contacts, clay lenses, and buried oxbow stream cutoffs. Trending heterogeneity commonly occurs in sedimentary formations of deltaic, alluvial, and glacial origin.

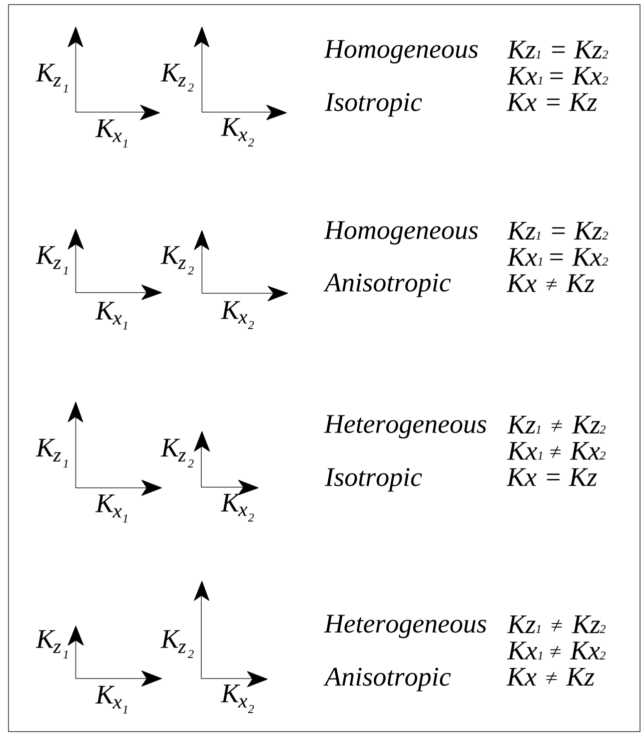

Figure 2-6. Homogeneity and isotropy

2-14. Flow in Stratified Media

a. General. Flow through stratified media can be described through the definition of a hydraulically equivalent conductivity (or effective hydraulic conductivity). Expressions for horizontal and vertical equivalent conductivities can be generalized from expressions developed for flow through porous media comprised of three parallel homogeneous, isotropic strata (Figure 2-7).

b. Horizontal flow. Horizontal flow through the media is given by Darcy's Law,

$$Q_x = K_x A_x \frac{\Delta h_T}{x} \tag{2-10}$$

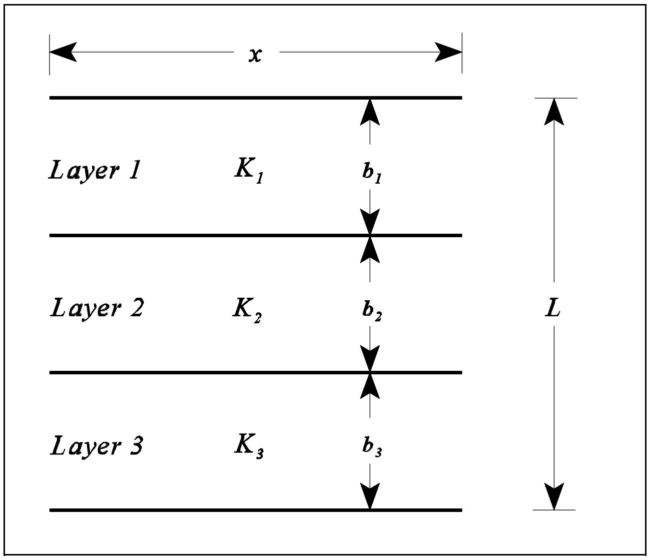

Figure 2-7. Stratified media

where

Δh_T = total hydraulic head drop across flow distance x

For the case of *i*, the strata method gives the expression for the *horizontal equivalent hydraulic conductivity* as:

$$K_x = \frac{\sum K_i b_i}{L} \tag{2-11}$$

c. Vertical flow. Similarly, vertical flow is given by Darcy's Law as:

$$Q_z = K_z A_z \frac{\Delta h_T}{L} \tag{2-12}$$

For the case of *i*, the strata method gives the expression for the *vertical equivalent hydraulic conductivity* as:

$$K_z = \frac{L}{\sum \frac{b_i}{K_i}} \tag{2-13}$$

2-15. Aquifer Storage

a. Storage coefficient. The *storage coefficient*, or *storativity S* is the volume of water that a permeable unit will absorb or expel from storage per unit surface area per unit change in head. At the water table, water is released from storage by gravity drainage. Below the water table, water is released from storage due to the release of hydrostatic pressure within the pore spaces which accompanies the withdrawal of water from the aquifer. The total load above an aquifer is supported by a combination of the solids skeleton of the aquifer and by the hydraulic pressure exerted by the water in the aquifer. Withdrawal of water from the aquifer results in a decline in the pore water pressure and subsequently more of the load must be supported by the solids skeleton. As a result, the rock particles are distorted and the skeleton is compressed, leading to a reduction in effective porosity. Additionally, the decreased water pressure causes the pore water to expand. Compression of the skeleton and expansion of the pore water both cause water to be expelled from the aquifer.

b. Specific storage. The *specific storage* S_s is the amount of water per unit volume of a saturated formation that is stored or expelled from storage owing to compression and expansion of the mineral skeleton and the pore water per unit change in hydraulic head. The specific storage (1/L) is given by:

$$S_s = \rho_w g(\alpha + n\beta) \tag{2-14}$$

where

ρ_w = density of water $[ML^{-3}T^{-2}]$

g = acceleration of gravity $[LT^{-2}]$

α = compressibility of the aquifer skeleton $[1/(ML^{-1}T^{-2}]$

n = porosity

β = compressibility of water $[1/(ML^{-1}T^{-2}]$

From field data, Helm (1975) estimated the specific storage of sands and gravels as 1×10^{-6} ft^{-1} and clays and silts as 3.5×10^{-6} ft^{-1}.

c. Storage coefficient of a confined aquifer. Within a confined aquifer the full thickness of the aquifer remains saturated when water is released or stored. Therefore, all water is released due to the compaction of the skeleton and expansion of the pore water and the storage coefficient (dimensionless) is given as:

$$S = bS_s \tag{2-15}$$

where

b = thickness of the aquifer [L]

Values of storage coefficient in confined aquifers are generally less than 0.005 (Todd 1980). Values between 0.005 and 0.10 generally indicate a leaky confined aquifer.

d. Storage coefficient of an unconfined aquifer. Within an unconfined aquifer the level of saturation varies as water is added to or removed from the aquifer. As the water table falls, water is released by gravity drainage plus compaction of the skeleton and expansion of the pore water. The volume of water released by gravity drainage is given by the specific yield of the aquifer. The storage coefficient of an unconfined aquifer is therefore given by the sum of the specific yield and the volume of water released due to the specific storage as:

$$S = S_y + hS_s \tag{2-16}$$

The value of specific storage is typically very small, generally less than 1 x 10^{-4} ft^{-1}. As the value of specific yield is usually several orders of magnitude greater than specific storage, the storage coefficient of an unconfined aquifer approximates its specific yield. The storage coefficient of unconfined aquifers typically ranges from 0.10 to 0.30. Estimates of specific yield for various deposits can be found in Johnson (1967).

e. Volumetric drainage. The volume of water drained from an aquifer due to a lowering of the hydraulic head can be computed from:

$$V_w = SA\Delta h \tag{2-17}$$

where

V_w = volume of water drained from aquifer [L^3]

S = storage coefficient (dimensionless)

A = surface area overlying the drained aquifer [L^2]

Δh = average decline in hydraulic head [L]

2-16. General Flow Equations

a. Confined aquifer. The governing flow equation for confined aquifers is developed from application of the law of mass conservation (continuity principle) to the elemental volume shown in Figure 2-8. Continuity is given by:

Rate of mass accumulation = Rate of mass inflow - Rate of mass outflow (2-18)

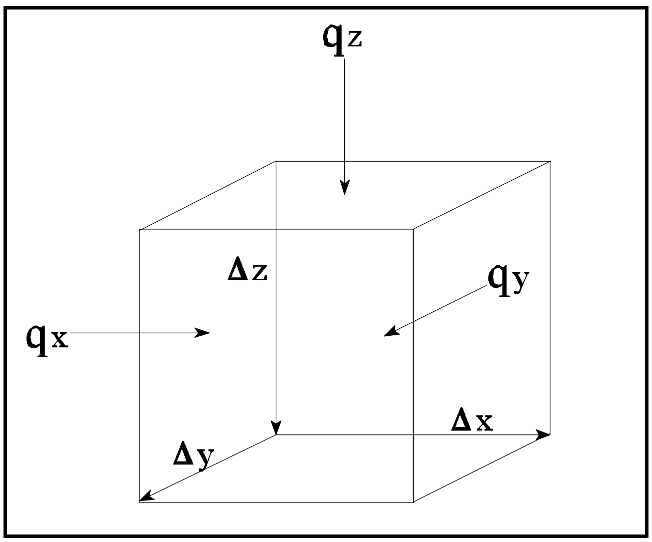

Figure 2-8. Elemental control volume

Integrating the conservation of mass (under constant density) with Darcy's Law, the general flow equation in three dimensions for a heterogeneous anisotropic material is derived:

$$S_s \frac{\partial h}{\partial t} = \frac{\partial}{\partial x}\left(K_x \frac{\partial h}{\partial x} \right) + \frac{\partial}{\partial y}\left(K_y \frac{\partial h}{\partial y} \right) + \frac{\partial}{\partial z}\left(K_z \frac{\partial h}{\partial z} \right) \tag{2-19}$$

Equation 2-19 is the general flow equation in three dimensions for a heterogeneous anisotropic material. Discharge (from a pumping well, etc.) or recharge to or from the control volume is represented as volumetric flux per unit volume ($L^3/T/L^3 = 1/T$):

$$S_s \frac{\partial h}{\partial t} + W = \frac{\partial}{\partial x}\left(K_x \frac{\partial h}{\partial x} \right) + \frac{\partial}{\partial y}\left(K_y \frac{\partial h}{\partial y} \right) + \frac{\partial}{\partial z}\left(K_z \frac{\partial h}{\partial z} \right) \tag{2-20}$$

where

W = volumetric flux per unit volume [1/T]

Assuming that the material is homogeneous, i.e. K does not vary with position, Equation 2-19 can be written as:

$$S_s \frac{\partial h}{\partial t} = K_x \frac{\partial}{\partial x}\left(\frac{\partial h}{\partial x} \right) + K_y \frac{\partial}{\partial y}\left(\frac{\partial h}{\partial y} \right) + K_z \frac{\partial}{\partial z}\left(\frac{\partial h}{\partial z} \right) \tag{2-21}$$

If the material is both homogeneous and isotropic, i.e. $K_x = K_y = K_z$, then Equation 2-21 becomes:

$$S_s \frac{\partial h}{\partial t} = K\left[\frac{\partial}{\partial x}\left(\frac{\partial h}{\partial x} \right) + \frac{\partial}{\partial y}\left(\frac{\partial h}{\partial y} \right) + \frac{\partial}{\partial z}\left(\frac{\partial h}{\partial z} \right) \right]$$

or, combining partial derivatives:

$$S_s \frac{\partial h}{\partial t} = K\left[\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} + \frac{\partial^2 h}{\partial z^2} \right] \tag{2-22}$$

Using the definitions for storage coefficient, ($S = bS_s$), and transmissivity, ($T = Kb$), where b is the aquifer thickness, Equation 2-22 becomes:

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} + \frac{\partial^2 h}{\partial z^2} = \frac{S}{T}\frac{\partial h}{\partial t} \tag{2-23}$$

If the flow is steady-state, the hydraulic head does not vary with time and Equation 2-23 becomes:

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} + \frac{\partial^2 h}{\partial z^2} = 0 \tag{2-24}$$

Equation 2-24 is known as the *Laplace equation*.

b. Unconfined aquifer. In an unconfined aquifer, the saturated thickness of the aquifer changes with time as the hydraulic head changes. Therefore, the ability of the aquifer to transmit water (the transmissivity) is not constant:

$$\frac{\partial}{\partial x}\left(K_x h \frac{\partial h}{\partial x} \right) + \frac{\partial}{\partial y}\left(K_y h \frac{\partial h}{\partial y} \right) + \frac{\partial}{\partial z}\left(K_z h \frac{\partial h}{\partial z} \right) = S_y \frac{\partial h}{\partial t} \tag{2-25}$$

where

S_y = specific yield [dimensionless]

For a homogeneous, isotropic aquifer, the general equation governing unconfined flow is known as the *Boussinesq equation* and is given by:

$$\frac{\partial}{\partial x}\left(h \frac{\partial h}{\partial x} \right) + \frac{\partial}{\partial y}\left(h \frac{\partial h}{\partial y} \right) + \frac{\partial}{\partial z}\left(h \frac{\partial h}{\partial z} \right) = \frac{S_y}{K}\frac{\partial h}{\partial t} \tag{2-26}$$

If the change in the elevation of the water table is small in comparison to the saturated thickness of the aquifer, the variable thickness h can be replaced with an average thickness b that is assumed to be constant over the aquifer. Equation 2-26 can then be linearized to the form:

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} + \frac{\partial^2 h}{\partial z^2} = \frac{S_y}{Kb}\frac{\partial h}{\partial t} \tag{2-27}$$

2-17. Aquifer Diffusivity

Aquifer diffusivity is a term commonly used in surface water/groundwater interaction and is defined as the ratio of transmissivity to storage coefficient (T/S). Equation 2-27 can be written as:

$$\frac{T}{S}\left(\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} + \frac{\partial^2 h}{\partial z^2}\right) = \frac{\partial h}{\partial t} \tag{2-28}$$

where

S = storage coefficient [dimensionless]

Equation 2-28 demonstrates the direct relationship between the promulgation of a groundwater flood wave (and pressure wave) and aquifer diffusivity. Equation 2-28 is applicable to homogeneous, isotropic aquifers under either confined or unconfined (where the change in aquifer thickness is insignificant) conditions.

2-18. Flow Lines and Flow Nets

a. Definition. Two-dimensional, steady flow which can be described by the Laplace equation (Equation 2-24) can be solved by a graphical construction of a flow net. A flow net is a network of curves called streamlines and equipotential lines. A streamline is an imaginary line that traces the path that a particle of groundwater would follow as it flows through an aquifer. In an isotropic aquifer, streamlines are perpendicular to equipotential lines. If there is anisotropy in the plane of flow, then the streamlines will cross the equipotential lines at an angle dictated by the degree of anisotropy. An equipotential line represents locations of equal potentiometric head (Section 2-6). A flow net is a family of equipotential lines with sufficient orthogonal flow lines drawn so that a pattern of square figures (or elements) results. While different elements may be different in size, the change in flow and change in hydraulic head is the same for all elements. Except in cases of the most simple geometry, the figures will not truly be squares.

b. Boundary conditions.

(1) All boundary conditions of the flow domain must be known prior to the construction of the flow net. Three types of boundary conditions are possible: a no-flow boundary, a constant-head boundary, and a water-table boundary. Along a no-flow boundary streamlines will run parallel, and equipotential lines will intersect the boundary at right angles. A constant-head boundary (such as large lake) represents an equipotential line and streamlines will intersect at a right angle while adjacent equipotential lines will run parallel to the boundary.

(2) A flow net is presented in Figure 2-9 for the two-dimensional, steady-state flow in a homogeneous, isotropic aquifer between two reservoirs with different hydraulic heads.

c. Analysis of results. The completed flow net can be used to determine the quantity of water flowing through the domain. For the system shown in Figure 2-9, the flow per unit thickness in one element is:

$$q_1 = K\Delta z \frac{\Delta h}{\Delta x} \tag{2-29}$$

But $\Delta x = \Delta z$, so Equation 2-29 becomes:

$$q_1 = K\Delta h = K\frac{\Delta h_T}{N_d} \tag{2-30}$$

where

N_d = the number of equipotential drops

The total flow per unit thickness is equal to the sum of the flow through each flow tube:

$$q_T = q_1 + q_2 \tag{2-31}$$

Since the flow through each flow tube is equal, $q_1 = q_2$ and:

$$q_T = N_f q_1 \tag{2-32}$$

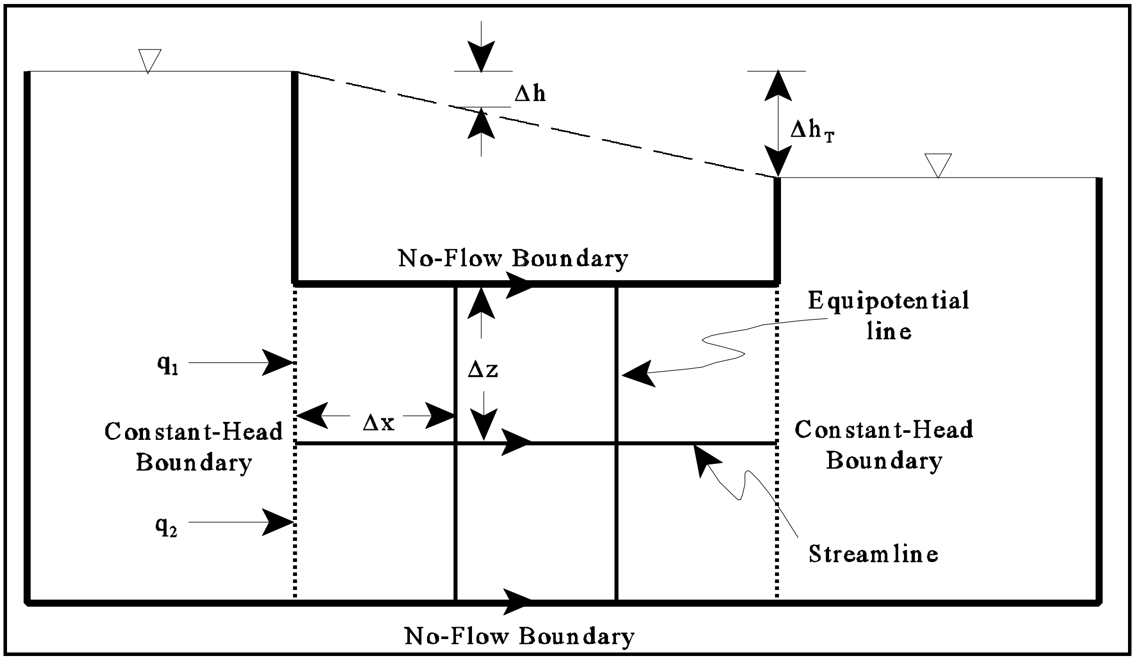

Figure 2-9. Basic flow net

where

N_f = the number of flow tubes in a domain

Substituting Equation 2-30 into Equation 2-32 gives:

$$q_T = K \Delta h_T \frac{N_f}{N_d} \tag{2-33}$$

Thus, the flow per unit thickness, as well as total flow, through this simplified system can be derived as a function of hydraulic gradient (K), the drop in hydraulic head (Δh_t), and the number of equipotential drops (N_d) and flow tubes (N_f) in the domain.

2-19. Estimating Capture Zones of Pumping Wells

a. General. A capture zone consists of the upgradient and downgradient areas that will drain into a pumping well. If the water table (or potentiometric surface) is flat, the capture zone is circular. However, in most cases the water table (or potentiometric surface) is sloping. Calculating capture zones of wells aids in the design of pump-and-treat groundwater remediation systems, and well-head protection zones. Figure 2-10 illustrates a typical capture zone. This section presents analytical methods for estimating the capture zones of pumping wells under steady-state conditions. Steady-state conditions are approximated after the well has been pumping for some time, and the change in water levels (with time) in the well and its zone of influence are judged insignificant. Assumptions included in the methods presented are as follows:

(1) The aquifer is homogeneous, isotropic, and infinite in horizontal extent.

(2) Uniform flow (steady-state) conditions prevail.

(3) A confined aquifer has uniform transmissivity and no leakage.

(4) An unconfined aquifer has a horizontal lower confining layer with no leakage and no recharge from precipitation.

(5) Vertical gradients are negligible.

(6) The well is screened through the full saturated thickness of the aquifer and pumps at a constant rate.

b. Confined steady-state flow. Assume the well in Figure 2-10 is located at the origin (0,0) of the x,y

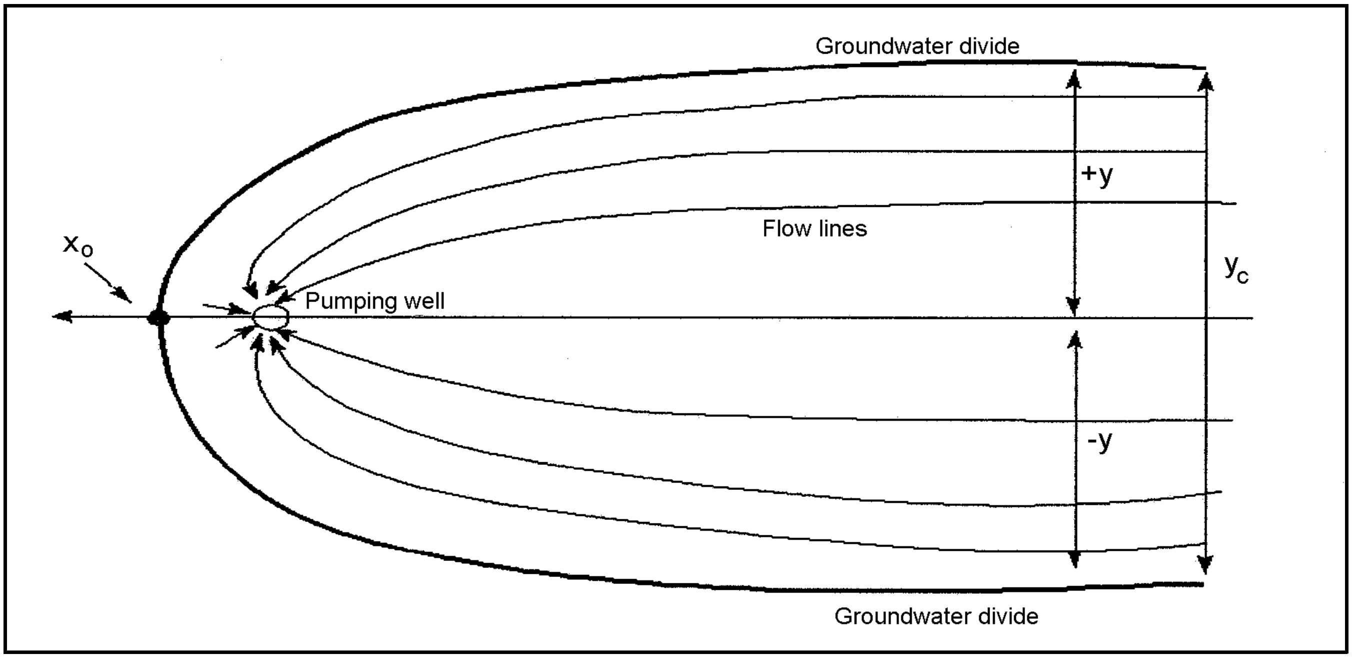

Figure 2-10. Capture zone of a pumping well in plan view. The well is located at the origin (0,0) of the x,y plane

plane. The equation to describe the edge of the capture zone (groundwater divide) for a confined aquifer when steady-state conditions have been reached is (Grubb 1993):

$$x = \frac{-y}{\tan(2\pi Kbiy/Q)} \qquad (2\text{-}34)$$

where

Q = pumping rate [L^3/T]

K = hydraulic conductivity [L/T]

b = aquifer thickness [L]

i = hydraulic gradient of the flow field in the absence of the pumping well (dh/dx) and tan (*) is in radians.

The distance from the pumping well downstream to the stagnation point that marks the end of the capture zone is given by:

$$x_0 = -Q/(2\pi Kbi) \qquad (2\text{-}35)$$

The maximum width of the capture zone as the distance (x) upgradient from the pumping well approaches infinity is given by (Todd 1980):

$$y_c = Q/Kbi \qquad (2\text{-}36)$$

c. Unconfined steady-state flow. Assume the well in Figure 2-10 is located at the origin (0,0) of the x,y plane. The equation to describe the edge of the capture zone (groundwater divide) for an unconfined aquifer when steady-state conditions have been reached is (Grubb 1993):

$$x = \frac{-y}{\tan[\pi K(h_1^2 - h_2^2)y/QL]} \qquad (2\text{-}37)$$

where

Q = pumping rate [L^3/T]

K = hydraulic conductivity [L/T]

h_1 = upgradient head above lower boundary of aquifer prior to pumping

h_2 = downgradient head above lower boundary of aquifer prior to pumping

L = distance between h_1 and h_2 and tan (*) is in radians.

The distance from the pumping well downstream to the stagnation point that marks the end of the capture zone is given by:

$$x_0 = -QL/(\pi K(h_1^2 - h_2^2)) \qquad (2\text{-}38)$$

The maximum width of the capture zone as the distance (x) upgradient from the pumping well approaches infinity is given by:

$$y_c = 2(QL)/(K(h_1^2 - h_2^2)) \qquad (2\text{-}39)$$

d. Example problem.

(1) Background. City planners are concerned about the potential contaminants from a toxic spill to contaminate the municipal water supply. The municipal well (which pumps at 19,250 m^3/day) is screened in a confined aquifer located 30 m to 80 m below the surface. Aquifer materials consist of coarse sands with a hydraulic conductivity of about 80 m/day. The well has been pumping for several years and conditions approach steady state. Groundwater flow in the aquifer trends toward the north. The potentiometric surface of the aquifer (measured before pumping commenced) drops approximately 1 m for every 200 m. The location of the well relative to the toxic spill is illustrated by Figure 2-11. Estimate the spatial extent of the capture zone of the pumping well.

(2) Solution. We are given:

the aquifer is confined
K = 80 m/day
b = 50 m
i = 1/200 = 0.005
Q = 19,250 m^3/day

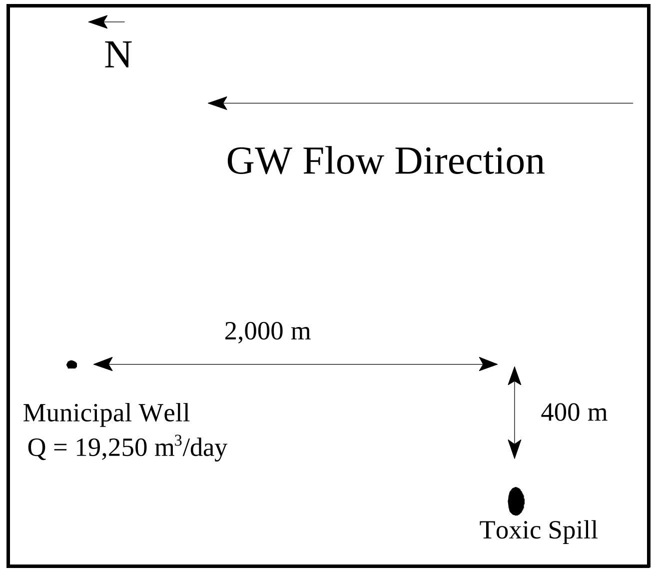

Figure 2-11. Location of toxic spill relative to pumping well

- Find maximum extent of capture zone:

  $y_c = Q/Kbi = 19250/(80)(50)(0.005) = 960$ m or ± 480 m from the x axis.

- Find location of stagnation point:

  $x_0 = - y_c/2\pi = -960/2\pi = - 150$ m

- Delineate boundary of the capture zone:

$$x = \frac{-\ y}{\tan(2\pi Kbiy/Q)} = -\ y/\tan(0.0065y)$$

±y	x
1	-150
100	-130
200	-50
300	120
400	670
450	2,045
480	24,000

(3) Analysis of results. The capture zone at a distance x = 2,000 m from the pumping well extends 450 m from the horizontal (x) axis. Therefore, initial calculations indicate that a portion of the contaminant plume will contaminate the municipal water supply unless mitigative measures are taken. Further investigation is warranted.

2-20. Specialized Flow Conditions

a. General. Darcy's Law (Section 2-11) is an empirical relationship which is only valid under the assumption of constant density and laminar flow. These assumptions are not always met in nature. Flow conditions in which Darcy's Law is not necessarily applicable are cited below.

(1) Fractured flow. Fractured-rock aquifers occur in environments in which the flow of water is primarily through fractures, joints, faults, or bedding planes which have not been significantly enlarged by dissolution. Fracturing adds secondary porosity to a soil medium that already has some original porosity. The original porosity consists of pores that are roughly similar in length and width. These pores interconnect to form a tortuous water network for groundwater flow. Fractured porosity is significantly different. The fractures consist of pathways that are much greater in length than width. These pathways provide conduits for groundwater flow that are much less tortuous than the original porosity. At a local scale, fractured rock can be extremely heterogeneous. Effective permeability of crystalline rock typically decreases by two or three orders of magnitude in the first thousand feet below ground surface, as the number of fractures decrease or close under increased lithostatic load (Smith and Wheatcraft 1992).

(2) Karst aquifers. Karst aquifers occur in environments where all or most of the flow of water is through joints, faults, bedding planes, pores, cavities, conduits, and caves, any or all of which have been significantly enlarged by dissolution. Effective porosity in karst environments is mostly tertiary, where secondary porosity is modified by dissolution through pores, bedding planes, fractures, conduits, and caves. Karst aquifers are generally highly anisotropic and heterogeneous. Flow in karst aquifers is often fast and turbulent where Darcy's law rarely applies. Solution channels leading to high permeability are favored in areas where topographic, bedding, or jointing features promote flow localization which focuses the solvent action of circulating groundwater, or well-connected pathways exist between recharge and discharge zones, favoring higher groundwater velocities (Smith and Wheatcraft 1992).

(3) Permafrost. Temperatures significantly below 0° C are required to produce permafrost. The depth and location of frozen water within the soil depends upon many factors such as fluid pressure, salt content of the pore water, the grain size distribution of the soil, soil mineralogy, and the soil structure. The presence of frozen or partially frozen groundwater has a tremendous effect upon flow. As water freezes, it expands to fill pore spaces. Soil that normally conveys water easily becomes an aquitard or aquiclude when frozen. The flow of water in permafrost regions is analogous to fractured flow environments where flow is confined to conduits in which complete freezing has not taken place.

(4) Variable-density flow. Unlike aquifers containing constant-density water, where flow is controlled only by the hydraulic head gradient and the hydraulic conductivity, variable-density flow is also affected by change in the spatial location within the aquifer. Water density is commonly affected by temperature, or total dissolved solids. As water temperature increases, its density decreases. A temperature gradient across an area influences the measurements of hydraulic head and the corresponding hydraulic gradient. Intrinsic hydraulic conductivity is also a function of water temperature (Equation 2-6). Thus, it is important to assess effects of fluid density on hydraulic gradient and hydraulic gradient in all site investigations.

(5) Saltwater intrusion. Due to different concentrations of dissolved solids, the density of the saline water is greater than the density of fresh water. In aquifers hydraulically connected to the ocean, a significant density difference occurs which can discourage mixing of waters and result in an interface between salt water and sea water. The depth of this interface can be estimated by the Ghyben-Herzberg relationship (Figure 2-12, Equation 2-40).

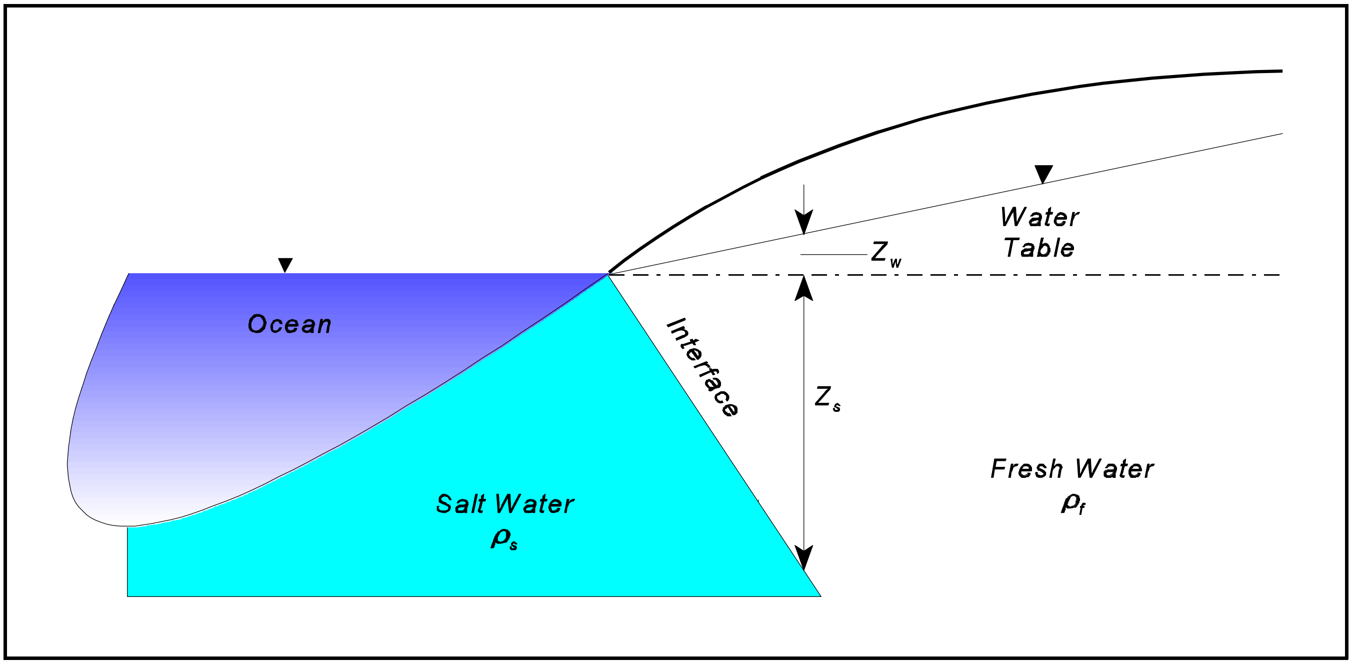

Figure 2-12. Saltwater-freshwater interface in an unconfined coastal aquifer

$$z_s = \frac{\rho_f}{\rho_s - \rho_f} z_w \qquad (2\text{-}40)$$

where

z_s = depth of interface below sea level

z_w = elevation of water table above sea level

ρ_s = saltwater density

ρ_w = freshwater density

For the common values of ρ_w = 1.0 and ρ_s = 1.25,

$$z_s = 40 z_w \qquad (2\text{-}41)$$

The Ghyben-Herzberg relationship assumes hydrostatic conditions in a homogeneous, unconfined aquifer. Additionally, it assumes a sharp interface between fresh water and salt water. In reality, there tends to be a mixing of salt water and fresh water in a zone of diffusion around the interface. If the aquifer is subject to hydraulic head fluctuations caused by tides, the zone of mixed water will be enlarged.

Chapter 3
Planning a Groundwater Investigation and Modeling Study

3-1. General

a. This chapter will present basic guidelines for performing a site characterization study, integrating hydrogeologic information into a conceptual model, performing simple analytical procedures, and formulating a computer model of groundwater flow. Specific attention is given to site reconnaissance, initial data interpretation, data acquisition, the formulation of conceptual and numerical/computer models, and guidelines for project management and personnel requirements. Additional information on performing an investigation study can be found in EC 1110-2-287 (1995), and U.S. Geological Survey (1977). Chapter 4 presents an overview of field investigation methods. Chapter 5 presents an overview on the technical aspects of computer modeling of groundwater flow.

b. In order to properly plan a hydrogeologic site investigation, the purpose of the investigation, the general geologic and hydrologic characteristics of the site, and the management constraints under which the investigation is to take place (financial and time restraints, availability of necessary equipment, availability of expertise) should be well understood by all involved in the project. Subsurface investigations are a dynamic and inexact science. The ability of the data acquired to provide an increasingly accurate representation of the hydrogeologic system increases with time, money, and the expertise of the specialists involved. Thus, the success of a groundwater investigation relies not only on the technical expertise of the specialists involved, but also on the effectiveness and efficiency of project management.

c. Examples of groundwater investigations related to Corps projects include the following:

(1) Contaminant remediation.

(2) Well production.

(3) Infiltration of runoff to the subsurface.

(4) Baseflow between aquifers and fixed bodies including streams and reservoirs.

(5) Effects of aquifer pumping on adjacent lakes and streams.

(6) Well installation involved with seawater infiltration barriers.

(7) Estimating grouting requirements.

(8) Dewatering of an excavation for construction purposes.

(9) Groundwater and surface water interrelated projects.

d. Groundwater investigations are based on the creation of an accurate conceptual model. A conceptual model is a simplified description of the groundwater system to be studied. The conceptual model can serve as a basis for formulating a numerical model. Due to recent advances in computer technology, the use of numerical models has been increasingly commonplace for predicting site reactions to defined stresses. A simplified flowchart that summarizes the general steps involved in a hydrogeologic site characterization and potential modeling study is presented as Figure 3-1.

3-2. Steps Involved in a Hydrogeologic Site Investigation and Potential Modeling Study

Hydrogeologic investigations generally are complex and require expertise from a number of different fields. Developing a plan that coordinates all the aspects of an investigation is vital to the success of the project. Study results can be derived from simplified analytical methods (as presented in Chapters 2 and 6) or a more complex numerical model which requires the use of computers (as presented in Chapter 5). Accuracy of the final product relies on efficient use of time, money, and personnel. The plan generally consists of a detailed outline of the objectives, scope, level of detail, procedures, available equipment, existing and potential problems, necessary data, well-thought-out schedule of tasks, and the resources available. Specific deliverables and milestones are defined. All personnel should

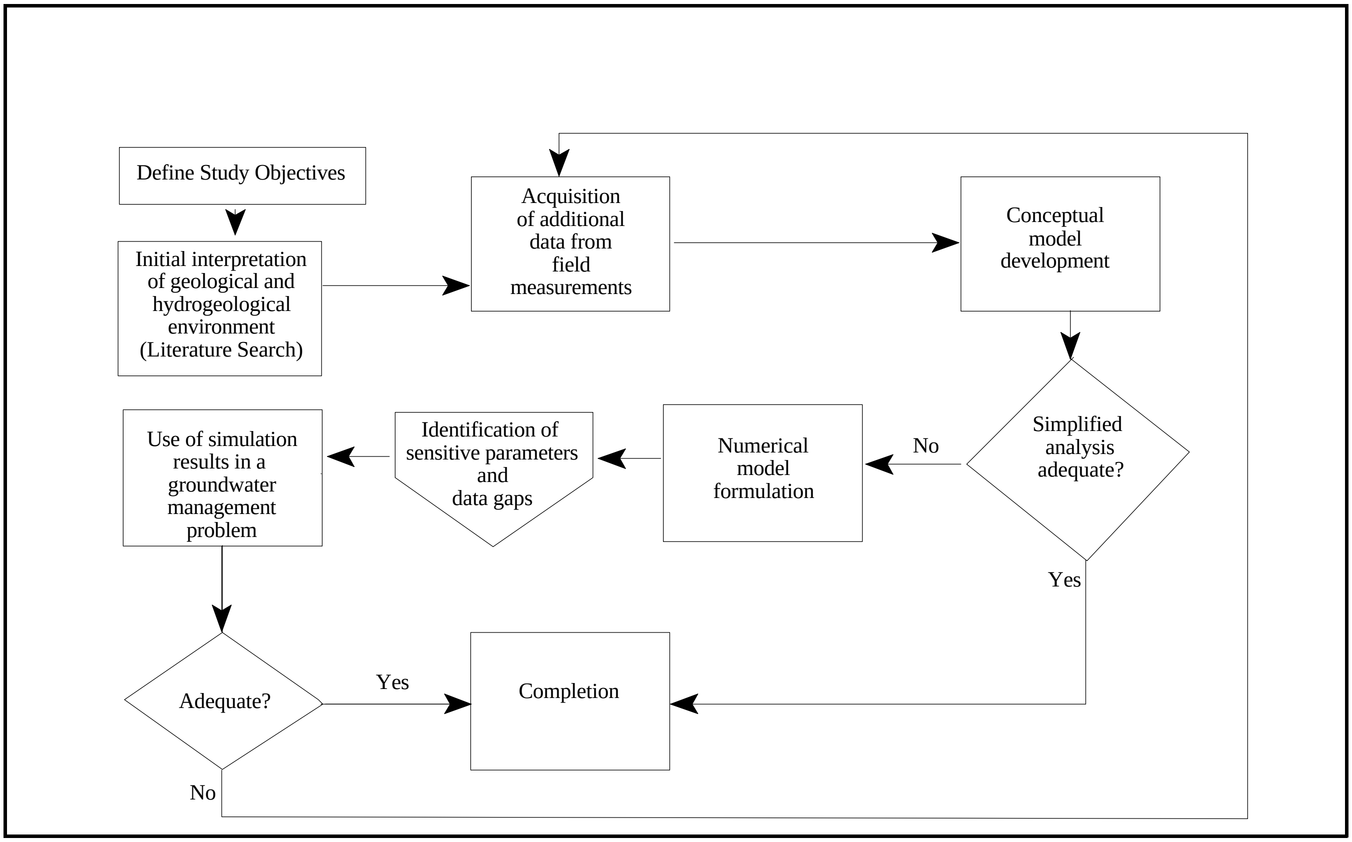

Figure 3-1. General flow diagram for characterizing a groundwater flow system for a management decision-making process

be well-informed of the importance of each job in relation to others.

a. Determining study objectives.

(1) Objectives defined. Objectives for a groundwater site investigation and analysis consist of a series of statements defining, as specifically as possible, the intended use and nature of the results being sought. The objectives define what answers are being sought. Defining sound and clear objectives early in the study is not as easy as it sounds. In some cases, simple analytical hand calculations will sufficiently address study objectives. In other cases, a more in-depth numerical analysis using computers will be required. When customers do not understand the capabilities and limitations of analytical procedures and groundwater models, and the study managers do not fully understand the needs of the customer, the work may begin with only a general idea of their objectives. This may lead to unmet expectations and waste of work time. It benefits the project manager, the technical assistance team, and the customer to take the time at the beginning of the study to define study objectives and also to define performance criteria.

(2) Desired specificity. Part of the study objectives statement should contain specific and descriptive constraints on what is expected (see below).

(a) The overall purpose of this study is to determine if reservoir inflows will significantly decrease as a result of upstream pumping at proposed irrigation and water supply wells. Groundwater modeling is desired to determine the maximum changes in water levels resulting from proposed pumping of up to 1,000 m^3/min from three wells located 200 m north of the stream. The wells are located 2,000 m, 5,000 m, and 7,000 m upstream of the reservoir inflow. The analysis should be performed for both the average seasonal high and seasonal low conditions. Perform the analysis using both a best-estimate approach and a worst-case approach that considers the uncertainty in the range of expected values of aquifer hydraulic conductivity and storage coefficient, streambed conductance, and recharge from precipitation. If significant reduction in streamflow is predicted, repeat the analysis for well locations of 300 m and 700 m north of the river.

(b) The overall purpose of this study is to determine groundwater flow travel times and directions beneath Swan Lake Landfill. Pollution from the landfill may contaminate nearby supply wells. Estimate the groundwater pore velocities and flow directions in the unconfined aquifer beneath the Swan Lake Landfill at the northeast property corner, the southeast property corner, and at the midpoint between. Estimate pore velocities directly down-gradient from these three points every 50 m until intersecting the western property boundary of the landfill. Estimate overall groundwater travel times from these three points to the western property boundary in the form of "best estimate, and expected ± 30 percent confidence interval values reflecting uncertainties in hydraulic conductivities and boundary conditions.

(3) Objectives and code selections. Modeling objectives should be set before conceptual model development and numerical code selection. Differing objectives lead to different modeling approaches. For example, data for a particular site may indicate that the water table elevations vary seasonally and interact with the levels of a nearby reservoir. If modeling objectives included determining the effects of reservoir level changes on the water table throughout the year, a conceptual model may describe the changing water table in a step-wise manner (e.g., monthly averages). This approach would lead to the construction of a model with time-varying boundary conditions and would require a software code with this capability. However, if the modeling objective is to determine effects only during that period of low reservoir levels, a simpler conceptual model describing only the typical low reservoir conditions may suffice. This latter case would allow construction of a much simpler steady-state numerical model. Additional information on proper code selection is presented in Chapter 5.

(4) Performance criteria. Performance criteria are set early in the modeling study and specify standards to measure the appropriateness of data acquisition, modeling approach, model construction, calibration, use, and presentation. Examples of criteria include: the basis for zones of homogeneous aquifer properties, limits on the magnitude of allowable calibration residuals, number and location of calibration targets, and type and degree of sensitivity analysis testing. These serve as a basis for evaluating model performance. Criteria are developed and agreed upon between the modeling team and the customer. In certain cases, a high degree of unknowns prior to the modeling effort requires the specification of criteria as model development progresses.

b. Initial interpretation of geologic/hydrologic environment.

(1) Field reconnaissance. A site investigation should be scheduled for all relevant personnel involved in the project. Field reconnaissance will provide a more complete understanding of site hydrogeology and project objectives. Additionally, it will aid in the determination of the feasibility of proposed methods and use of equipment. Objectives which should be documented by photographs and a trip log include the following:

(a) General character of local geology.

(b) Prominent topographic features.

(c) Location and flow rates of wells and adequacy of local wellhead protection.

(d) Nature, volume, flow, and location of surface waters.

(e) Nature, volume, and location of any potential surface and sub-surface contamination.

(f) Nature and location of any significant impermeable areas.

(g) Nature and location of areas of significant vegetative ground cover.

(2) Literature search, accessing existing data. Existing data should be assessed completely as a first step to any hydrogeologic site investigation. Much of the data necessary for developing a conceptual model may already have been collected during previous investigations of the site. Geologic, hydrologic, geographic, and other data can be obtained from electronic databases and from reports by the Corps, the U.S. Geological Survey, other federal agencies, and state, local, and private organizations. The level of detail desired will also affect the data needs. All data should be critically reviewed to validate their accuracy and applicability to investigation purposes. Data that should be reviewed include the following:

(a) Regional hydrogeologic reports.

(b) Previous investigations of aquifer and/or surface waters.

(c) Available information on groundwater use, including purpose, quantities, and future projections.

(d) Boring log data.

(e) Cone penetrometer log data.

(f) Monitoring well data.

(g) Production well data.

(h) Well construction characteristics.

(i) Geophysical data.

(j) Geologic, hydrologic, and topographic maps and cross sections of study area.

(k) Aerial photographs.

(l) Land use maps.

(m) Soil maps.

(n) Long-term climatic data.

c. Acquisition of additional data. Data requirements should be assessed by the investigator as a function of cost and the level of acceptable uncertainty associated with the particular groundwater investigation. Such recommendations will be consistent with the level of detail required. The investigator should choose the investigative methods which provide the most valuable data within time and cost constraints. The following is a list of some available sources.

(1) Geologic information.

(a) Surficial structures and deposits.

(b) Drilling samples.

(2) Hydrologic data.

(a) Distribution of groundwater levels (horizontally and vertically).

(b) Flow to/from wells.

(c) Slug tests.

(d) Laboratory tests of drilling samples.

(e) Response of groundwater levels to fluctuations in surface water.

(f) Response of groundwater levels to loading events.

(g) Water chemistry (geochemistry and isotope hydrology).

(h) Artificial tracers.

(3) Geophysical data.

(a) Borehole.

(b) Surface methods.

(c) Ground penetrating radar.

(d) Cone penetrometers.

d. Conceptual model development.

(1) Definition. A conceptual model is a simplified description of the groundwater system to be

studied. Development of a conceptual model is the most important step in developing a computer model. Natural area boundaries, hydrostratigraphy, water budget, aquifer properties, potentiometric surfaces and other features are described in a level of detail commensurate with the ability of the data to represent the system. In other words, a highly heterogeneous system requires more (and/or higher quality) data to provide for the same level of detail in representing a more homogeneous system. Features often described in conceptual models include the following:

(a) Relationship and extent of hydrogeologic units (hydrostratigraphy, hydrofacies).

(b) Aquifer material properties (porosity, hydraulic conductivity, storativity, isotropy).

(c) Potentiometric surfaces.

(d) Water budget (inflows and outflows such as: surface infiltration, lateral boundary flux, leakage through confining units, withdrawals and injections).

(e) Boundary locations (depth to bedrock, impermeable layer boundaries, etc.).

(f) Boundary conditions (fluxes, heads, natural water bodies).

(g) System stresses (withdrawal wells, infiltration trenches, etc.).

(h) Dynamic relationships varying through time.

(i) Water chemistry (varies with purpose; drinking, irrigation, pumping, etc.).

(2) Data requirements.

(a) What is the physical extent of the system to be studied (horizontally and vertically)?

(b) What are the distinct measurable components of the system?

(c) What data are currently available?

(d) Does some of the available data add little value toward meeting study objectives?

(e) What aspects of the conceptual model lack adequate definition?

(f) If data are not available for a particular feature, is a computer model expected to be sensitive to that feature?

(3) Integrated interpretation. During conceptual model formulation, technical specialists perform an integrated interpretation of all the data available to produce the most accurate assessment of site conditions (Figure 3-2). The integrated approach to a site characterization assesses data from multiple sources, and combines the data to produce a more accurate interpretation of the site characteristics. The accuracy of the model will depend on the accuracy of the available data, the time frame in which the results were collected, and the expertise of the specialists in combining and interpreting the data.

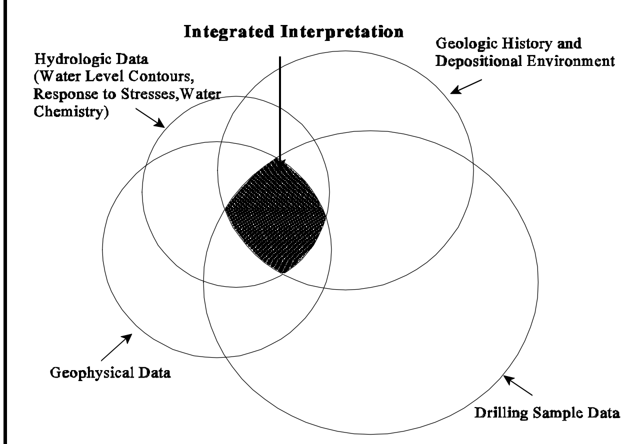

Figure 3-2. Example of an integrated approach to a site characterization study

(4) Presentation of conceptual model. Graphic descriptions of the conceptual model can include simplified hydrogeologic cross sections,

potentiometric surface maps, structure maps of hydrogeologic units, three-dimensional graphics, and schematic water balance diagrams. Graphics should complement a written description.

(a) Topographic maps. Topographic maps aid in delineating drainage areas, locating desired cross sections, and locating boundaries for other maps (including geologic, depth to water, flow gradients, recharge and discharge areas, and other related features). Many different scales are available ranging from 1:126,700 to 1:4,800.

(b) Aerial photographs. Aerial photos are often used as substitutes for topographic maps. Multiple photographs may be used with a stereoscope to obtain a three-dimensional view of the area. The Department of Agriculture and the U.S. Geological Survey are good sources for these photographs.

(c) Geologic maps and sections. These maps are helpful when complex geologic structures and variances occur. When accompanied by analysis reports, they aid in locating aquifers, water level conditions, structural and stratigraphic control of water movement, and other related factors. The U.S. Geological Survey is the primary source of such materials, although mining companies, universities, and other geology-related organizations may also be helpful in locating a map and/or study of a specific area.

(d) Water table contour maps. These maps are similar to topographic maps, the difference being that they show water table elevations as opposed to ground elevations.

(e) Piezometric surface maps. Piezometric surface maps are similar to water table contour maps, except they are based on the piezometric potential developed in piezometers which penetrate a single confined aquifer.

(f) Depth-to-water maps. These maps show the depth from the ground surface to the water table. Care should be taken when using these maps due to their condition-specific accuracy. They are usually developed using a limited number of reference points during a specific condition. Thus, fluctuations may occur through time, and misleading data could arise.

(g) Cross-sectional maps. These maps are developed using borehole data. The vertical stratigraphy of the subsurface is mapped out using multiple boreholes spaced in a horizontally planar manner. Water table, aquifer, and other variances in the subsurface can be identified and mapped out.

(h) Fence diagrams. These maps are similar to cross-sectional maps, except they illustrate the surface and subsurface in three dimensions.

(i) Hydrographs. Hydrographs show water level changes for individual wells over time.

(j) Water budget. A description of the inflows and outflows (water budget) through the model area boundaries is key to the conceptual model. Recharge through the upper surface, leakage through the lower surface, and flux through the sides of the model area should be estimated using methods from hydraulic and seepage theory. These estimates are required to ensure that predicted and calculated flux through model boundaries match to an acceptable degree. Examples of inflows into the study area include: subsurface flow from upgradient of the study area, recharge from precipitation, leakage from surface water, and injection wells. Examples of outflows from the study area include: subsurface flow downgradient of the study area, evapotranspiration, leakage to surface water, and extraction wells.

e. Conceptual model use. The conceptual model, together with the computer modeling objectives, helps guide code selection, grid design, boundary condition and time variation designation, and setting of initial conditions in the numerical model. For example, if the conceptual model describes a simple horizontal flow system in an unconfined aquifer above a relatively flat impermeable layer, construction of a simple two-dimensional mathematical model would likely suffice. If, however, the conceptual model reveals a much more complex system, then a quasi three-dimensional or fully three-dimensional model should be considered. The conceptual model also helps to identify data needs.

f. Uncertainty in conceptual models. Formulation of the conceptual model requires dealing with uncertainty. Most of the conceptual model components can be represented either as single values, ranges of values, or as statistical distributions. Where possible, the values should be carried through the analysis as ranges or distributions. This is particularly the case for hydraulic conductivity estimates, the variation of which has a relatively large impact on modeling results. The computer modeler's approach in dealing with uncertainty depends on the quality and completeness of available data, the level of confidence required by the modeling objectives, and the quality and quantity of resources available to do the modeling job. Eliminating uncertainty altogether is an unlikely and impractical objective for groundwater modeling. Managing uncertainty and communicating its effects are essential to good modeling. During conceptual model development, the following should be considered when managing uncertainty:

(1) Document the quality, quantity, and completeness of the data upon which the model is based.

(2) Document data sources.

(3) Assess additional data needs.

(4) Document the boundary condition assumptions.

(5) If a component is set at a single value, or if a hydrofacie surface is set as unique, document the assumptions and implications of doing so.

(6) If components are to be carried forward to the numerical model as a range or a distribution, document how these were derived and why doing so meets the modeling objectives.

g. Simplified analysis. In cases where data are limited or a detailed analysis is not required, study objectives can often be met without the use of computer models. In these cases, simplified analytical calculations (see Chapters 2 and 6), such as estimating the capture zone of a pumping well or using Darcy's Law to estimate groundwater flow volumes, may adequately address study objectives. In cases when a computer model is required, initial analytical analyses should be performed for comparison with simulated results.

h. Numerical model development. Once a complete conceptual model has been developed, a numerical model can be generated. Parameters determined during conceptual model development are integrated into a computer model. The computer model is then calibrated to reproduce measured field conditions. If model calibration is judged acceptable, the computer model can be used to predict other hydrogeologic changes due to new stresses; for instance, the introduction of a pumping well for groundwater cleanup, within the site. If the numerical model does not produce acceptable results during calibration, then it may be necessary to completely reanalyze the geologic and hydrogeologic parameters. A complete discussion of numerical model development is presented in Chapter 5.

3-3. Project Management

a. General. A successful modeling analysis involves much more than manipulating modeling software. The analysis should develop and present information that answers the questions posed by the project. This requires proper planning and control.

b. Planning and control. Project management involves project planning and project control. Because groundwater models often play a decisive role in water supply and remediation studies, the modeler should be part of the project management team and take part in setting the objectives and schedules in the project management plan.

(1) Project planning. Project planning includes identification of existing data and data needs, definition of work requirements, work time frame, and resources needed. Definition of work requirements, primarily in the form of establishing modeling objectives, is the single most important key to project planning for groundwater modeling studies. Before constructing the conceptual model and before choosing the modeling software, groundwater modelers and their customers need to understand what exactly the objectives of performing a modeling study are. An understanding of

modeling objectives and some experience with modeling is also necessary when estimating expected schedule and resource requirements. There is no “one size fits all” schedule and technical resource requirements in groundwater modeling. Simple models can be performed in a matter of weeks while complex models often require months or even years.

(2) Project control. Project control includes monitoring progress, measuring performance, and making adjustments as necessary. Key intermediate milestones that can be used to monitor progress are:

(a) Establishment of modeling objectives.

(b) Completion of a conceptual model.

(c) Successful calibration of the model application.

(d) Obtaining preliminary results.

(e) Model testing.

(f) Obtaining final results and reporting.

Measuring the quality of model development and making appropriate adjustments to the work scope are not easy tasks. Typically, a modeler requires outside review to aid in assessing the adequacy of model development as it progresses. At a minimum, peer review should be performed at the completion of the conceptual model and then again when interpreting preliminary results even if the modeler feels that things are going well.

c. Maintaining/developing corps technical expertise. Performing groundwater modeling studies “in-house” provides the advantage of development of expertise within the Corps. Such expertise greatly increases the Corps’ capacity to write scopes of work that give specific and complete direction for both in-house and contracted work. Expertise within the Corps also adds flexibility for performing small projects as well as controlling changes in large projects. Where many changes are likely to occur, contracting out without adequate control can lead to excessive costs. When it is necessary to contract out groundwater modeling projects, scope of work specifications should be detailed and closely reviewed by Corps personnel having a high degree of expertise. In Corps offices where expertise in groundwater modeling is limited, personnel could be sent to several groundwater modeling courses which are available. Corps centers of expertise and research and development centers can assist in providing direction and review.

3-4. Personnel

a. Project team. A groundwater modeling project truly requires a multidisciplinary approach. Although the modeler performs a key role, also required are the project manager, geologists and/or hydrogeologists, geotechnical engineers, field data collection personnel, hardware and software specialists, and graphics support personnel. In addition, interfaces with hydrologists and hydraulic engineers, meteorologists, chemists, soil engineers/physicists, Geographical Information Systems specialists, environmental engineers and data management specialists are often required. The basic modeling team should consist of the following positions:

(1) Project manager.

(2) Groundwater modeler.

(3) Geologist, hydrogeologist.

(4) Software/graphics support.

(5) Peer review.

A single individual can perform more than one of the above roles. It is important that the modeler maintain close relationships with geologists and other field personnel who characterize the site. Arranging for offsite peer review increases the soundness of the modeling approach and adds credibility to the final product.

b. Roles.

(1) Project manager. The project manager is responsible for overall planning and control. The setting of objectives, schedules, and allocation of

resources require early planning by the project manager. Oversight of funds and interaction with various larger project elements, customers, and regulatory agencies are main activities. The project manager also monitors progress and actively participates in making corrective actions to the scope and schedule. The project manager is also responsible for the assembly of the modeling team and delegating specific project assignments and responsibilities.

(2) Groundwater modeler. The groundwater modeler is responsible for developing the conceptual model, designing the model grid, determining parameter inputs, model calibration, model execution, and interpretation of results. Because of the highly subjective nature of the modeling process, it is important that the modeler have a strong background in hydrogeology, along with an intimate understanding of site geology.

(3) Geologist/hydrogeologist. The responsibilities of the geologist can include direct gathering of field data, interpretation of site characterization information, and development of the conceptual model with the modeler.

(4) Software and graphics support. These personnel provide assistance in code installation and testing, linking programs, and developing formulation of output. Graphics support is important in model documentation and presentation of model results.

(5) Peer review. A highly qualified expert in the field of groundwater modeling should be retained to provide periodic technical oversight along with critical review of the final model. Review of the conceptual model is essential. To provide a different perspective in the modeling effort, it is helpful that this person be somewhat removed from the daily modeling project effort.

3-5. Example of Site Characterization and Modeling Process

This example outlines simplified steps in performing a groundwater investigation and modeling study. An overview of the technical aspects of groundwater modeling is presented in Chapter 5.

a. Define objectives. A feasibility study was performed for construction of a wastewater treatment plant that will discharge up to 5 million gallons per day to an aquifer recharge lagoon. Groundwater modeling was used to: (1) make predictions of the highest water table elevations expected in a typical 10-year time period resulting from maximum natural and wastewater recharges, and (2) make predictions of the groundwater flow paths for the same conditions. Specific detailed statements defining modeling objectives were agreed upon by the parties involved.

b. Form study team. To perform this task, a study team was assembled consisting of a project manager, a geologist, a groundwater modeler/ hydrogeologist, a hydrologist, and a graphics support technician; several of these tasks can be performed by the same person. A knowledgeable modeler from another District agreed to provide peer review. The team defined the modeling objectives, schedule, and milestones for tracking progress.

c. Develop conceptual model. A conceptual understanding of site hydrogeology was initially compiled from several existing well logs, water table measurements, climate records, a topographic map, and field observations. These sources indicated several data gaps that led to the installation of three additional wells and performance of a pumping test. Using the additional data gathered, a conceptual model was developed that described the hydrogeologic units, hydrogeologic boundaries, water budget, existing water table surface, and aquifer transmissivities.

d. Simplified analysis. Darcy's Law (see Chapter 2) was applied to provide an initial estimate of seepage rates from the recharge lagoon to the groundwater table.

e. Select computer model. Having defined the study objectives and groundwater flow system, modeling software with the capability of computing water table elevations and flow paths was then selected. The software had the capability to perform a two-dimensional, steady-state approach that was determined to be consistent with the modeling objectives.

f. Develop the initial model input files. At this point the features of the conceptual model were transferred to the modeling software input file where, with help from the software, they were represented on a two-dimensional grid. This grid was superimposed on a site map which aided the assignment of transmissivities and recharge locations. The grid structure and zonation of aquifer properties was kept as simple as possible; i.e., the complexity of the model should be commensurate with the ability of the data to represent the system. The boundaries of the grid required special attention to ensure that flows and heads within the model area realistically matched those of the surrounding areas.

g. Model calibration. After a software input file containing the "best estimate" representation of the natural system was created, the modeling software was run repeatedly to resolve any obvious anomalies. This process of iteratively running the modeling software, comparing the output to observed site conditions, then adjusting inputs within specified ranges is called calibration and was continued until the computed heads and boundary flows matched field conditions.

h. Sensitivity analysis. Sensitivity analysis is the measure of uncertainty in the calibrated model caused by uncertainty in aquifer parameters and boundary conditions. Sensitivity analysis was performed by systematically changing the calibrated values of hydraulic parameters by defined factors (such as 0.5 and 2.0), while holding all other model parameters constant. Sensitivity analysis identifies parameters most important in conceptual model development, and can be used as a guide for additional data acquisition.

i. History matching. Model simulations were run and results were compared with field data which were not used in the calibration process. For example, if the model was calibrated to simulate seasonal water level fluctuations for 1994, a comparison between measured and simulated water levels from 1993 can be performed if data are available. A favorable comparison lends greater validity to model predictive capability. An unfavorable comparison indicates that further calibration (and perhaps data acquisition) is necessary.

j. Model application. The calibrated model was then used to simulate the "worst case" condition (highest water table elevations expected) that included maximum wastewater discharge plus natural recharge from the 10-year storm. The highest typical seasonal water table condition and the lowest estimates of transmissivities for the hydrogeologic units were also used.

k. Produce modeling results. The modeling software produced output in the form of water table maps and flow-line graphics. When compared with observed conditions, the simulated output presented expected changes in groundwater flow directions as a result of wastewater plant discharge. The simulations demonstrated that several low-lying areas at the site may be inundated by raised groundwater levels for the conditions tested.

l. Final report. The modeling study report documented the purpose, approach, and results of the analysis. The report fully addressed the following topics:

(1) Definition of model objectives.

(2) Data acquisition.

(3) Summary of geologic and hydrologic conditions.

(4) Development of conceptual model.

(5) Computer code selection.

(6) Definition of model grid and model layer.

(7) Boundary and initial conditions.

(8) Determination of hydrogeologic properties (calibration).

(9) Sensitivity analysis.

(10) Model application.

(11) Interpretation of results.

(12) Recommendations for future monitoring and data gathering.

A complete explanation of the physical basis or other justification for all parameters used in model development was included. Additionally, a discussion of how uncertainty was dealt with was included in the modeling report. (Approaches for dealing with uncertainty are discussed in Chapter 5). Tabular comparisons of computed and observed values used for calibration and for results of the sensitivity analysis were included. Graphical representation, including color graphics, were included for conveying the complexities of spatially distributed information. Time-varying graphics in the form of video presentations were provided.

m. Post audit. Following development of the initial numerical model, the predictive capability of the model was periodically monitored as new field data became available. Model recalibration should be performed if it is judged that the additional data allow for a significantly more accurate conceptualization of site conditions.

3-6. Conclusion

The purpose of this chapter was to present a general overview of performing a groundwater site investigation and modeling study from a project management perspective. The following chapters provide technical information on investigative methods, and numerical modeling of groundwater flow. Additionally, a detailed summary of a specific groundwater site investigation and modeling study is presented as Appendix C.

Chapter 4
Field Investigative Methods

4-1. General

a. Adequate conceptualization of a hydrogeologic system often requires the acquisition of new field data. This chapter provides an overview of different methods which can be employed to gain a better understanding of subsurface conditions pertaining to the occurrence and flow of groundwater. Key references are provided to allow for a more detailed understanding of concepts and applications. Hazardous, toxic, and/or radioactive waste (HTRW) investigations often require special consideration beyond the scope of this text.

b. Initially, information that can be obtained in the process of, and as a product of, the construction of a well is described. The construction and development of wells can provide a wealth of information on subsurface conditions. Geologic logging during drilling of a borehole enables the delineation of high-conductivity and low-conductivity strata. Borehole geophysical methods can provide information on the lithology, porosity, moisture content, permeability, and specific yield of water-bearing rocks; additionally, borehole geophysical methods can also help define the source movement and chemical characteristics of groundwater. Completed wells offer information on hydraulic head and water quality. Finally, wells provide a conduit through which stress can be placed upon an aquifer by the extraction or injection of water. Aquifer properties, such as transmissivity and storage coefficient, can then be estimated by the aquifer response to these stresses.

c. An overview of surface geophysical methods is then presented. Surface geophysical methods allow for the nonintrusive gathering of information on subsurface stratigraphy and hydrogeologic conditions. Surface geophysical methods include seismic refraction and reflection, electrical resistivity, gravitational methods, electromagnetic methods, and ground-penetrating radar. A section on cone penetrometers is then included. Cone penetrometers often provide a cost-effective method for gathering significant data on subsurface stratigraphy. Finally, overviews on the use of geochemistry, and the response of water levels to loading events to gain information on subsurface conditions are included.

d. An additional method for acquiring new hydrologic data is studying the interaction between surface water and groundwater. For example, the effects of surface water fluctuations on groundwater levels can be used to estimate the aquifer transmissivity and storage coefficient. Analytical methods for quantifying interaction of surface water and groundwater are presented in Chapter 6.

4-2. Wells

a. Well drilling methods.

(1) General. The overriding objectives in pumping well design and construction are as follows: the attainment of the highest yield possible with minimum drawdown in pumping wells, good water quality, minimizing environmental effects, ensuring borehole integrity, minimizing siltation, and reasonable short- and long-term costs. Various well drilling methods have been developed in response to the range of geologic conditions encountered, and the variety of borehole depths and diameters that are required. The most common methods employed in drilling deep wells are direct and reverse circulation mud rotary, direct and reverse circulation air-rotary with casing drive, hollow stem auger drilling, and the cable tool method. The terms direct and reverse refer to the direction in which the drilling fluid (mud or air) is circulated. In direct drilling, the drilling fluid is circulated down the string of drill tools out the bit and up the annulus between the tool string and the borehole wall. In reverse, as the name implies, the direction of circulation is reverse that of direct drilling. An in-depth description of drilling methods can be found in Driscoll (1986).

(2) Mud rotary. The rotary methods provide a rapid means for drilling in a wide range of geologic conditions. In direct mud rotary, a hollow rotating bit is used, through which a mixture of clay and water, known as drilling mud, is forced out under pressurized conditions. This drilling mud serves the dual purpose of transporting cuttings to the surface along with sealing the borehole wall, thus allowing the hydrostatic

pressure of the drilling mud to hold the borehole open. Advantages in using the direct mud rotary method include its rapid drilling rate, and the non-requirement for placing casing during drilling operations in unconsolidated material. Disadvantages include mud disposal, the need to remove mud lining from the boring walls during well development, and difficulty in identifying when the water table is encountered.

(3) Air rotary. In air-rotary drilling, air, rather then drilling mud, is used to remove cuttings and cool the bit. Air rotary drilling can be done open-hole (semi- and consolidated formations) or in conjunction with simultaneously driving the casing (unconsolidated formation). Air for drilling is supplied either by an on-board or auxiliary air compressor. Air is circulated at volumes up to 57 m^3/min at pressure up to 2,400 kpa; however, in unconsolidated formations pressure above 1,000 kpa is unnecessary and can cause excessive borehole erosion and borehole instability. The air should be filtered to remove compressor oil and other contaminants prior to use in drilling. When drilling in unconsolidated formations, air rotary drilling is typically done in conjunction with driving the casing to stabilize the borehole. The advantages of air rotary drilling are its rapid drilling (penetration) rate, lack of drilling mud and associated clean-up, and the accuracy with which the water table can be located when drilling at low pressures (i.e., < 700 kpa). Disadvantages include higher cost, access for larger equipment, and noise.

(4) Hollow stem auger. Hollow stem auger drilling is a rotary drilling method that does not require circulation of a fluid medium. Rather, the borehole is advanced and cuttings removed by a cutter head followed by a continuous flight or helix of auger ramps which can be likened to a wood screw. Modern hollow stem auger drills can install wells to depths greater than 80 m in unconsolidated formation (hollow stem augers are not for use in semi- or consolidated formations). When drilling, a cutting head is attached to the first auger flight, and as the auger is rotated downward, additional auger flights are attached, one at a time, to the upper end of the previous auger flight. As the augers are advanced downward, the cuttings move upward along the continuous flighting. The hollow stem or core of the auger allows drill rods and samplers to be inserted through the center of the augers. The hollow stem of the augers also acts to temporarily case the borehole, so that the well screen and casing may be inserted down through the center of the augers once the desired depth is reached, minimizing the risk of possible collapse of the borehole that might occur if it is necessary to withdraw the augers completely before installing the well casing and screen. The hollow-stem auger drilling technique is not without problems. These are more completely described in Aller et al. (1989), but generally include:

(a) Heaving: Sand and gravel heaving into the hollow stem may be difficult to control, and may necessitate adding water to the borehole.

(b) Smearing of silts and clays along the borehole wall: In geologic settings characterized by alternating sequences of sands, silts, and clays, the action of the augers during drilling may cause smearing of clays and silts into the sand zones, potentially resulting in a considerable decrease in aquifer hydraulic conductivity along the wall of the borehole. The smearing of clays and silts along the borehole wall may, depending on the site-specific properties of the geologic materials, significantly reduce well yield or produce unrepresentative groundwater samples even after the well has been developed.

(c) Management of drill cuttings: Control of contaminated drill cuttings is difficult with the auger method, especially when drilling below the water table.

(5) Cable tool method.

(a) The cable tool method is one of the oldest and most versatile drilling techniques. Penetration into the subsurface is achieved by lifting and dropping a string of tools suspended from a cable, with the weight of the falling tools providing the driving force. The string of tools generally consists of four sections: the swivel socket, the drilling jars, the drill stem, and the drill bit. The swivel socket rotates the bit, allowing it to strike a different area of the hole bottom with each stroke. The drilling jars consist of two loosely interconnected rods. Their purpose is to enable a reverse hammering effect to free the bit and stem, should they become lodged in the borehole. The drill stem keeps the drill bit driving straight, while also providing additional weight. The bit crushes and mixes any materials in the drilling path. The debris is removed by the addition of water (when above the water table) into the borehole to produce a slurry that can then be pumped out. Cable tool drilling is usually limited to borehole diameters less than 75 cm (30 in.) and drilling depths less than 600 m (2,000 ft).

(b) The advantages of this type of drilling are low cost and ability to drill into a variety of mediums in many conditions. Additionally, this method provides for an accurate logging of formation changes. It is sensitive to any medium changes, allowing the driller to adjust sample increments. This method also uses less water than other drilling methods, which is convenient when drilling in desolate arid regions. The major disadvantages are a slow drilling progress, the limitation in borehole sizes and depths, and the need to drive casing coincident with drilling when drilling in unconsolidated materials.

b. Well design and completion.

(1) General. Well design should address the following factors: the depth of the well screen or screens; diameter of screen and casing; type of material (e.g. mild steel, stainless steel, etc.); the type of well screen (mill slot, shutter slot, continuous slot, etc.); gradation of the filter pack (formation stabilizer) surrounding the well screen; and the type and composition of annular seals (e.g. conventional neat cement versus high-solids bentonite grout). Well completion involves setting and positioning casing and well screens, placing filter pack, sealing the annular space, and constructing well-head features at the ground surface. While each of these design elements is dependent upon site-specific conditions such as the purpose of the well and available funding, there are some general guidelines that need to be incorporated into every design. Figure 4-1 illustrates basic well components. Driscoll (1986) presents a more complete discussion of well design and completion procedures.

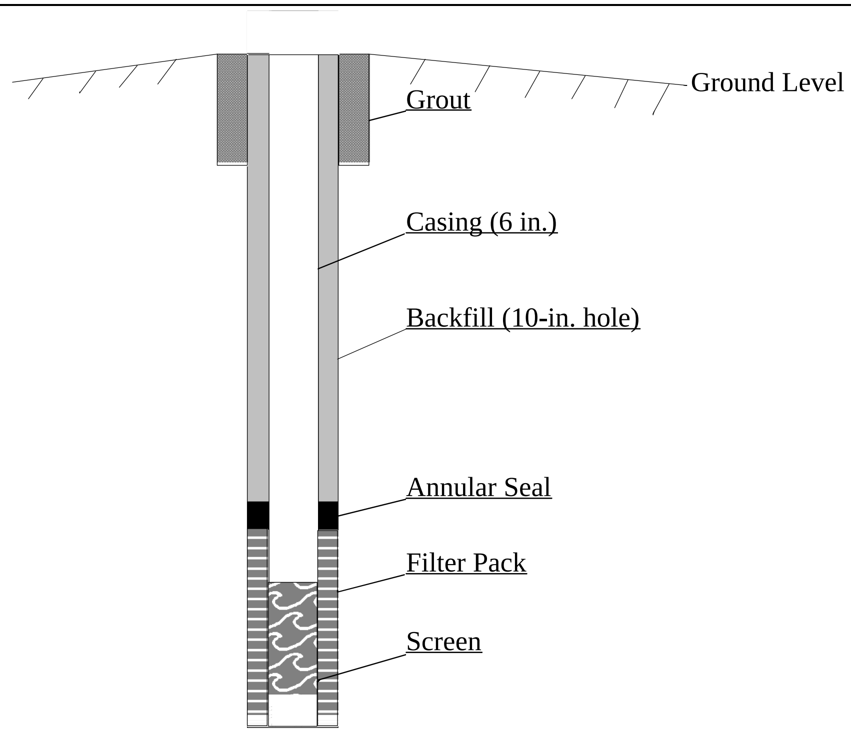

Figure 4-1. Basic well components

(2) Casing. Casing should be of sufficient strength to withstand not only the depth of installation, but also a certain amount of abuse during handling and installation. Casing should be of sufficient diameter to accept a pump at least one size larger than currently required in order to account for potential lowering of the water table.

(3) Filter pack. The filter pack commonly consists of a graded sand which is artificially placed around the well screen to stabilize the aquifer, minimize sediment entering the well, permit the use of a large screen slot size, and provide an annular zone of high permeability. The filter pack is a key element in the hydraulic efficiency of the well. The filter pack needs to provide a smooth gradation transition from the formation. Essentially, the gradation of the filter pack is based upon the uniformity coefficient (a measure of how well it is sorted) and the D_{70} (70 percent passing sieve size) of the formation. These parameters are obtained from sieve analyses of formation samples obtained during exploratory drilling. Depending upon the uniformity coefficient, the D_{70} of the formation is multiplied by a factor from 3 to 9. The resulting value is the new D_{70} for the filter pack. Utilizing the new D_{70}, the filter pack gradational curve is constructed such that it roughly parallels the formation gradational curve.

(4) Well screen. Well screen design encompasses a balance between required strength and desired hydraulic efficiency. Hydraulic efficiency is basically a function of the amount of open area in a well screen; the greater the open area, the greater the area available for groundwater flow and thus greater hydraulic efficiency. Generally, one strives to maximize hydraulic efficiency at a prescribed strength. A key element of well screen design is the size of the openings, referred to as slot size. The slot size is a function of the filter pack gradation. The slot size is typically selected to retain 80-90 percent of the filter pack. Well screens are placed at the depths of interest to: hydrologically isolate formations, prevent sand movement into the well, and minimize hydraulic resistance to water entering the well. Screens are available in a variety of materials, diameters, and slot sizes depending on the hydrologic and water quality parameters of the aquifer, the desired well yield, and aquifer thickness.

(5) Annular seals. In choosing an annular sealant, the following factors should be considered: borehole stability (e.g., an unstable or caving borehole needs an easily placed, quick-setting sealant such as high-solids bentonite grout); the method with which the well was drilled; and the type of well casing (e.g., the heat of hydration from thick cement seals can deform/melt PVC casing).

(6) Placement of cement or grout. Wells are cemented, or grouted, in the annular space surrounding the casing to prevent entrance of water of unsatisfactory quality, to protect the casing from corrosion, and to stabilize caving rock formations. It is important that the grout be introduced at the bottom of the space to be grouted by use of a tremie pipe to ensure the zone is properly sealed.

c. Well development. Wells are developed by removing the finer material from the natural formations surrounding the screening. A new well is developed to increase its specific capacity and prevent silting. Development procedures are varied and include pumping, surging, hydraulic jetting, and addition of chemicals. The basic purpose of all these methods is to agitate the finer material surrounding the well so that it can be carried into the well and pumped out. Pumping involves discharging water from a well in successive steps until clear water is produced. Surging utilizes a block which is moved in an up-and-down motion with increasingly faster strokes. Compressed air can also be utilized to create rapid changes in water levels within the well casing. Hydraulic jetting utilizes a high-velocity stream of water which is rotated across the full extent of the screened area removing finer-grained material from the gravel packing by turbulent flow. Chemical additives, such as hydrochloric acid, can be employed in open hole wells in limestone or dolomite formations to remove finer particles and widen fractures.

d. Well efficiency. The objective in well design is to avoid excessive energy costs by constructing a well that will yield the required water with the least drawdown. Well efficiency can be defined as the ratio of the drawdown in an aquifer at the radius of the well borehole (just outside the filter pack in the aquifer) to the drawdown inside the well. The difference between aquifer and well drawdowns is attributed to head losses as water moves from an aquifer into a well and up the well bore. These well losses can be reduced by reducing the entrance velocity of the water, which is accomplished by installing the maximum amount of screen and pumping at the lowest acceptable rate. Other factors involved in reducing well loss include proper development techniques and proper filter pack design.

4-3. Monitoring Wells

a. The primary objectives of a monitoring well are to provide an access point for measuring groundwater levels and to permit the procurement of groundwater samples that accurately represent in situ groundwater conditions at the specific point of sampling. To achieve these objectives, it is necessary to fulfill the following criteria:

(1) Construct the well with minimum disturbance to the formation.

(2) Construct the well with materials that are compatible with the anticipated chemical and geochemical environment.

(3) Properly complete the well in the desired zone.

(4) Adequately seal the well with materials that will not interfere with the collection of representative water samples.

(5) Sufficiently develop the well to remove any additives associated with drilling and provide unobstructed flow through the well (Aller et al. 1989).

b. In addition to appropriate construction details, the monitoring well must be designed in concert with the overall goals of the monitoring program. Key factors that must be considered include the following:

(1) Intended purpose of the well.

(2) Placement of the well to achieve accurate water levels and/or representative water quality samples.

(3) Adequate well diameter to accommodate appropriate tools for well development, aquifer testing equipment, and water quality sampling devices.

(4) Surface protection to assure no alteration of the structure or impairment of the data collected from the well (Aller et al. 1989).

c. In essence, one should strive to construct a well that is transparent to the aquifer in which it is constructed. Aller et al. (1989) and American Society for Testing and Materials (ASTM) (1993) provide in-depth guidelines for the design and installation of groundwater monitoring wells.

4-4. Geologic Logging

Logs of rock and soil encountered during drilling can provide the most direct and accurate means for the delineation of high-conductivity and low-conductivity strata. The character, thickness, and succession of the underlying formations provide important data as to existing aquifers, aquitards, and aquicludes and the interaction between surface water and the subsurface. All geologic logs should follow procedures listed in Engineer Manual (EM) 1110-1-4000 (1994).

4-5. Measuring Water Levels

a. Data uses. Accurate measurements of groundwater levels are essential for conceptualization of site hydrogeology. Information which can be provided by water level measurements includes the following:

(1) Rate and direction of groundwater movement.

(2) Status or change in groundwater storage.

(3) Change in water level due to groundwater withdrawal.

(4) Amount, source, area of recharge, and estimate of discharge.

(5) Hydraulic characteristics of an aquifer.

(6) Identify areas where the water table is near the land surface.

(7) Delineate reaches of losing or gaining streams or canals.

b. Data sources. Water level data can be acquired from a number of sources, including existing wells, piezometers, and from surface water/ groundwater interfaces such as lakes, streams, and springs. Observation wells can be installed at necessary locations where other resources do not exist.

c. Data requirements. In addition to water level elevation, the following information should be recorded with each measurement:

(1) Local well name and owner.

(2) Date drilled.

(3) Well use.

(4) Location by legal description, such as latitude and longitude coordinates.

(5) Approximate location relative to local landmarks.

(6) Elevation of land surface and measuring point.

(7) Well depth, size and type of casing, location and type of perforations.

d. Methodology. There are essentially three main techniques to measuring water levels in non-flowing wells, the graduated steel tape (wetted-tape method), the electrical measuring line, and air lines. All three have their advantages and disadvantages for measuring under certain conditions.

(1) Graduated steel tape method. This method is widely considered to be the most accurate method for measuring water levels in non-flowing wells. Tapes in lengths of 50, 100, and 300 m, and 100, 200, 500, and 1,000 ft are among the most common. They are available as either black or chromium-plated, with black being preferred by most. Tapes up to 150 m (500 ft) in length are usually hand-crank-operated, while longer tapes are often motor-driven. A lead weight is generally attached to the end to aid in plumbness and added feel. A lead weight is less likely to foul any pumps due to its soft nature. The attachment should be made so that should the weight become lodged in the well, it will break off allowing retrieval of the tape. To acquire a measurement, the lower end of the tape is marked with carpenter's chalk. The amount submerged into the water will enable a reading to be taken by viewing the wetted portion. Corrections for thermal expansion of tapes greater than 300 m (1,000 ft) in length should be applied in extreme temperatures. Two measurements should be taken, with an agreement of less than 0.6 cm (0.25 in.). If water is dripping down the well, or if the water surface is disturbed, it may be impossible to get an accurate reading. If oil is present on top of the water in depths greater than a foot, then the thickness of the oil layer must be known to compensate for the lower density; thus, a higher water level measurement. The oil level can be determined by using a water detector paste that will show both the water and the oil levels.

(2) Electrical method. Electrical measuring devices generally consist of two electrodes that complete a circuit when immersed in water. These electrodes are attached to a power supply by a conductive cable. There are various other types of electrode/cable combinations, with the two-conductor cable and special probe being the most common. The cable is generally 150 m (500 ft) long and uses a hand-cranked reel. The advantage to the electrode method is the ability to take multiple measurements without having to fully remove the cable from the well. It also is more accurate than the steel tape when measuring in a pumping well where the water may be splashing or dripping down the well. These conditions will usually foul a steel tape measurement. They are also safer when used in pumping wells because they detect the water immediately, lessening the chance of lowering the probe into pump impellers. The disadvantages are that they are more bulky than the steel tape, and less accurate under ideal conditions. The measurements should be within 1 cm (0.04 ft) for less than 60-m (200-ft) depths, and about 3 cm (0.1 ft) for 150-m (500-ft) depths. Measurements have been within 15 cm (0.5 ft) for depths as great as 600 m (2,000 ft). Adapters can be added to sensing probes to detect oil. After multiple uses, the length of the cable should be checked because stretching may occur during use.

(3) Air line. Air pressure lines consist of an airtight tube that when submerged into the water is purged by compressed air. The pressure required to purge the tube is related by the depth of the tube in the water. Multiplying the pressure in psi by 2.31 ft/psi will give the depth. In metric, multiplying the pressure in Pascals by 4,850 m/Pascal will give the depth. That distance can then be subtracted from the total length of the tube in the well and the depth to water will be determined. This technique works well where the surface of the water is being disturbed. The durability of air lines has historically been a problem, as they become clogged with mineral deposits or may form leaks, both leading to false measurements. The accuracy of this technique relies mostly on the accuracy of the gauge being used. Other measuring techniques should be employed periodically.

e. Recording devices. Automated devices for recording changes in water levels may be mechanical, electronic, or electromechanical. Electromechanical devices usually consist of a float that measures the actual vertical changes in water levels. Mechanical or electronic devices consist of submerged probes that measure changes in pressure from varying water depths. Rapid changes in depth are measured with greater accuracy with pressure sensing devices since they are able to detect the changes more rapidly than a float. Floats lose most of their accuracy from cable friction along the well walls. The recording device itself is generally a simple mechanism that is able to chart the water level versus time. Due to the delicate nature of the recording device, some sort of housing should be provided to protect it from weather and vandalism.

f. Measurement frequency. The basic factors determining measurement frequency are the types of fluctuations expected, the potential use of the data, and the available personnel. Fluctuations occur due to many factors, including: pumping, recharge (from any number of sources, manmade and natural), and evapotranspiration. Use of the data will determine the desired frequency of measurements, with restraints from equipment and personnel. Automatic recorders are best for high-frequency measurements. Human error may cause discrepancies in frequent measurements causing the data to skew results. Weekly and monthly measurements may miss pumping and recharge events completely. Under certain pretenses, infrequent measurements (semi-annual) may suffice.

g. Effect of changes in barometric pressure on water levels in confined aquifers. Changes in atmospheric pressure can have a significant effect on water levels in wells penetrating a confined aquifer. In confined aquifers, well measurements should be corrected to a constant barometric pressure (Section 4-12).

4-6. Pumping Tests

a. General. Pumping tests (or aquifer tests) are in situ methods that can be used to determine hydraulic parameters such as transmissivity, hydraulic conductivity, storage coefficient, specific capacity, and well efficiency. Hydrogeologic values derived from pumping tests are averaged over the spatial zone of influence of the test. The basic steps involved in performing a pumping test are: (1) background measurements; (2) pumping test measurements; and (3) recovery measurements. Depending on data needs and well and geological conditions, two general types of pumping tests can be performed : constant-rate pumping tests, and step-drawdown pumping tests. Data measured during a pumping test include: flow rates, time, and water levels. Atmospheric pressure measurements can be additionally made when performing tests in confined aquifers. Several analytical methods for data interpretation are available. Appendix D presents an overview of general methods available. Recommended references for a more in-depth discussion of pumping tests and accompanying analytical methods are: Dawson and Istok (1991), Kruseman and De Ridder (1983), Driscoll (1986), and Walton (1987).

b. Flow to pumping wells.

(1) General. The study of well hydraulics is a complicated blend of mathematics, fluid mechanics, and soil physics. It is as much an art as a science. The following sections present wells from a somewhat idealized perspective, oftentimes greatly simplifying the true system. Through this idealization, the resulting equations simplify to solutions that are exact or easily approximated to near exact solutions. General assumptions for all cases are: (a) that the aquifer is isotropic, homogeneous, and of infinite areal extent;

(b) the well fully penetrates the aquifer; (c) the flow is horizontal everywhere within the aquifer; (d) the well diameter is so small that storage within the well is negligible, and; (e) water pumped from the well is discharged immediately with decline of piezometric head. The general governing equation for all idealized cases is Laplace's equation in cylindrical coordinates. Detailed derivations of these equations are performed in Freeze and Cherry (1979).

(2) Specific capacity. The specific capacity of a well is the yield per unit drawdown, and is determined by dividing the pumping rate at any time by the drawdown at the same time. The specific capacity of a well depends both on the hydraulic characteristics of the aquifer and on the construction, pumping rate, and other features of the well. Values of specific capacity, available for many supply wells for which aquifer-test data are not available, are widely used by hydrologists to estimate transmissivity.

(3) Cone of depression. The movement of water from an aquifer into a well results in a cone of depression (also known as zone of influence). Because water must converge on the well from all directions, and because the area through which the flow occurs decreases toward the well, the hydraulic gradient must get steeper toward the well. The size of a cone of depression is dependent primarily on the well pumping rate, elapsed time since start of pumping, aquifer type, aquifer transmissivity, and aquifer storativity (Figure 4-2). Withdrawals from an unconfined aquifer result in drainage of water from rocks through which the water table declines as the cone of depression forms. Because the storage coefficient of an unconfined aquifer closely approximates the specific yield of the aquifer material, the cone of depression expands slowly. On the other hand, a lowering of the water table results in a decrease in aquifer transmissivity which will cause an increase in drawdown both in the well and in the aquifer. Withdrawal from a confined aquifer causes a drawdown in artesian pressure and a corresponding expansion of water and compression of the mineral skeleton of the aquifer. The very small storage coefficient of a confined aquifer results in the rapid expansion of the cone of depression.

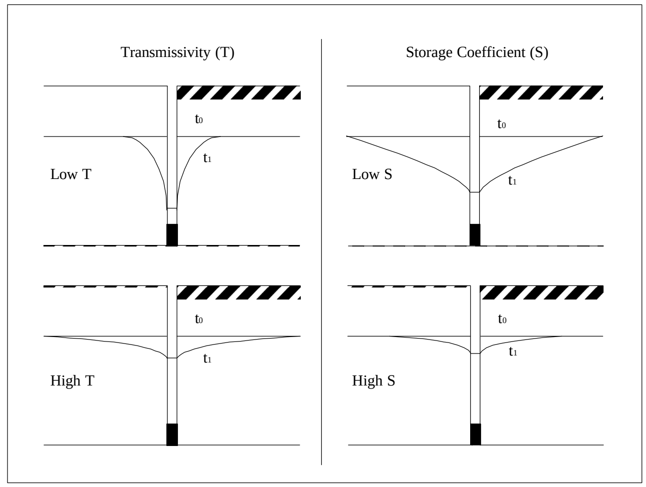

Figure 4-2. Influence of transmissivity and storage coefficients on cone of depression for similar aquifers at a constant pumping rate

c. Types of pumping tests.

(1) Constant-rate test. A constant-rate pumping test consists of pumping a well at a constant rate for a set period of time (usually 24 or 72 hr), and monitoring the response in at least one observation well. The number and location of observation wells is dependent upon the type of aquifer and the objectives of the study. Values of storage coefficient, transmissivity, hydraulic conductivity (if aquifer thickness is known), and specific capacity can be obtained.

(2) Step-drawdown test. During a step-drawdown test, the pumping rate is increased at regular intervals for short time periods. The typical step-drawdown test lasts between 6 and 12 hr, and consists of three or four pumping rates. Because step-drawdown pumping tests are typically much shorter than constant-rate pumping tests, transmissivity and storativity values are not as accurate for these tests. The primary value of the step-drawdown test is in determining the reduction of specific capacity of the well with increasing yields.

(3) Recovery test. A recovery test consists of measuring the rebound of water levels towards preexisting conditions immediately following pumping. The rate of recovery is a valuable source of data

which can be used for comparison and verification of initial pumping test results.

d. Pumping test design.

(1) General. Before implementing a constant-rate or step-drawdown pumping test, the well should be developed adequately to reduce the influence of well construction on aquifer response. Aquifer data from a pumping test should be derived from both the pumping well and appropriately placed observation wells. Small diameter pumping wells are preferable because of their quicker response to changes in hydraulic head. The accuracy of data taken from a pumping well is often less reliable because of turbulence created by the pump. Furthermore, drawdown data from an observation well are required for the accurate calculation of the storage coefficient of the aquifer. Thus, at least one observation well should be used when practicable. Design of a field pumping test (also called an aquifer test) is as much art as it is science, and requires judgement tempered by experience. Assumptions must be made concerning the type of aquifer and its characteristics, and a suitable test developed based on those assumptions. The following procedure may be followed as a guide to design an aquifer pumping test.

(2) Development of conceptual geologic model. To design a pumping test, it is necessary to have some knowledge (or make assumptions) of the subsurface stratigraphy. Items of concern include the type, thicknesses, and dip of strata, as well as the ease with which this strata can be drilled. If no borings have been drilled in the project area, it will be necessary to start with a geologic literature search of USGS and state agency documents (see Section 3-2).

(3) Development of conceptual hydrologic model. Items of concern include type and depth of the aquifer(s), as well as the hydraulic conductivity, transmissivity, storativity or specific yield, and yield and specific capacity of pumping wells. Water quality may also be a concern, particularly if a discharge permit is required for disposal of the pumped water. If no wells have been drilled in the project area, it will be necessary to glean this information from U.S. Geological Survey (USGS) or state agency reports, or to make assumptions that seem reasonable based on the conceptual geologic model. Nearby property owners may have wells and can be of some help, as can local water well drillers.

(4) Define the test objectives. While it may at first seem that the objectives are simply to “learn about the aquifer,” on further examination the question becomes “What exactly do you want to learn about the aquifer?” Is this test being conducted as part of a water budget study where the concern is defining transmissivity and storativity; or is the test part of a water supply study where the concern is specific capacity and safe well yield; is the test part of a groundwater contaminant transport study where the ultimate question is the velocity of the groundwater? Is there any concern between the possible interconnection of two or more separated aquifers, such as a near-surface water table aquifer and a deeper artesian aquifer? A careful definition of the test objectives is essential to ensure a successful test.

(5) Determining the well pumping rate (Q). It is usually desirable to pump at the maximum practical rate so as to stress the aquifer as much as practical for the duration of the test. This translates into more drawdown at the pump well and observation wells, and therefore more data available for the final analysis. The maximum rate will be limited by the efficiency of the well construction and the specific capacity of the well, and should be a rate such that the well will not be dewatered below the pump intake screen during the duration of the test. If a new well is to be drilled for this pump test, then it will be necessary to initially assume a pumping rate based on the conceptual hydrologic model previously mentioned.

(6) Determining the test duration (t). Practical constraints usually limit the time available for the test, and at a maximum it is useless to run the test beyond the point at which a steady-state condition is reached (i.e., no more drawdown) or the point at which the pumping well intake screen begins to dewater. Pumping tests last anywhere from 6 hr to 2 weeks, depending on the objectives and the aquifer characteristics, but most probably fall between 1 and 3 days for the pumping phase of the test, followed by an equal amount of time to monitor the recovery.

(7) Determining the observation well locations. Observation wells should be located in areas of influence of the pumping test. However, wells placed too close to the pumping well will be influenced by the vertical flows in the immediate vicinity of the pump well and may yield erroneous data. A good rule of thumb is to place the observation wells a distance no less than (1.5)(*b*) from the pumping well, where *b* is the aquifer thickness. However, this rule has often been violated with no apparent ill effects, especially for low pumping rates. To determine the maximum radial distance (*r*) at which observation wells can be placed from the pumping well, assume a minimal drawdown (*s*) that you believe to be significant, and solve the appropriate discharging well analytical equations in reverse. To check for aquifer anisotropy, locate wells at equal distances from the pumping well but in differing azimuthal directions. To allow for distance drawdown solutions and to allow for calculation of the cone of influence of the pumping well, locate wells at differing radial distances from the pumping well. Project budgets will usually provide a practical constraint for the number of observation wells, so well locations must be optimized to fit the test objectives, and compromises often must be made. In the event that an observation well(s) cannot be optimally located, then the observation well(s) should be replaced with a cluster of depth-staggered piezometers. A piezometer cluster would have at least one piezometer at (0.25)(*b*) and another at (0.75)(*b*). Using depth-staggered piezometers allows the collection of draw-down data which can be readily corrected for partial penetration and delayed yield.

(8) Drill the pumping well. Since the conceptual models developed earlier are not absolutes, it is often necessary to reevaluate and refine these models as actual field data are obtained. The first well drilled should be the pumping well, and it should be thoroughly logged as drilled to evaluate the actual site stratigraphy. A performance test should be conducted on this well as soon as possible after completion, and prior to drilling the observation wells. Down-hole tests may be conducted on the open hole prior to constructing the well to obtain hydrologic data on particular zones. These tests may consist of either pump-in (pressure tests) or pump-out (variable head) tests, and can be analyzed by methods as explained in U.S. Department of Interior (1977). These tests will yield data to refine the earlier estimates of specific capacity, well yield, transmissivity, hydraulic conductivity, and aquifer thickness.

(9) Refine the conceptual geologic and hydrologic models. Use the data obtained from the first well drilled to reevaluate the pumping rate, test duration, and observation well locations. Make changes to the field layout as needed. From a practical standpoint, this may have to be accomplished in a motel room at night after working all day in the field with the drilling crew.

(10) Drill the first observation well and perform a mini-pumping test. It would be most conservative to drill the closest observation well first, since this well will predictably have the greatest drawdown of all the planned observation wells. Use the refined conceptual models to predict drawdown in the single observation well after a short period of pumping (1 to 4 hr recommended). Measure drawdown in both the pumping well and the observation well, and compare the measured and predicted values. Further refine the conceptual models as necessary and drill the remaining observation wells.

e. Single well tests. It is also possible to obtain useful data from production wells when data from observation wells are not available. The procedure for this determination is similar to the Jacob method. Values of drawdown are recorded directly from the pumping well. However, because of well loss in the pumping well, the estimates of storativity and transmissivity derived from the straight-line intercept with the line of zero drawdown are a rough approximation.

f. Well interference. Well interference occurs when the cones of depression from adjoining wells intersect. Well interference reduces the available drawdown, and the maximum yield of a well.

g. Aquifer boundaries. Aquifer boundaries can be of two types: recharge and impermeable. A recharge boundary is a boundary which serves as a potential or actual source of recharge to the aquifer, and has the effect of decreasing the response of an aquifer to withdrawals. Examples of recharge boundaries include zones of contact between the aquifer and rivers, lakes, and mountain-front recharge areas. An impermeable boundary is a zone of contact across which minimal flow occurs. Impermeable boundaries have the effect of increasing the response of the aquifer to withdrawals. One of the assumptions of analytical methods used to analyze pump test data is that the aquifer to which they are applied is infinite in extent. This assumption is commonly met for practical purposes in aquifers that are aerially extensive to a degree where pumping will not have an appreciable effect on recharge and discharge, and most water is derived from groundwater storage. In situations where lateral boundaries have an appreciable influence on aquifer response, the hydraulic effect can be assumed, for analytical convenience, to be due to the presence of other pumping wells, called image wells. A recharge boundary has the same effect on drawdowns as a recharging image well, and an impermeable boundary has the same effect on drawdowns as a discharging image well.

4-7. Slug Tests

Slug tests are applicable to a wide range of geologic settings as well as small-diameter piezometers or observation wells, and in areas of low permeability where it would be difficult to conduct a pumping test. A slug test is performed by injecting or withdrawing a known volume of water or air from a well and measuring the aquifer's response by the rate at which the water level returns to equilibrium. Hydraulic conductivity values derived relate primarily to the horizontal conductivity. Slug tests have a much smaller zone of infiltration than pumping tests, and thus are only reliable at a much smaller scale. A general overview of slug tests can be found in Fetter (1994). Recommended references for in-depth discussions of slug tests and accompanying analytical methods are: Bouwer and Rice (1976); Bouwer (1989); Hvorslev (1951); Cooper, Bredehoeft, and Papadopulos (1967); and Papadopulos, Bredehoeft, and Cooper (1973).

4-8. Borehole Geophysics

a. General.

(1) Subsurface geophysical logging involves the lowering of a sensing device within a borehole for the determination of physical parameters of the adjacent rock and fluids contained in that rock. This is accomplished by the propagation or detection of electrical currents, radiation, thermal flow, or sound waves through the surrounding subsurface. Geophysical well logs can be interpreted to determine the lithology, geometry, resistivity, formation factor, bulk density, porosity, permeability, moisture content, and specific yield of water-bearing rocks, and to define the source, movement, and chemical and physical characteristics of groundwater. Borehole geophysical logs provide a continuous record of various natural or induced properties of subsurface strata and of the pore fluids contained within those strata. Borehole geophysics also provide information about the fluid standing within the borehole and well construction. These data, when interpreted in a conjunctive manner, can provide accurate and detailed information about subsurface conditions.

(2) A general overview of borehole geophysical methods is presented in this section. For a more in-depth discussion, the following references are recommended: EM 1110-1-1802, Keys and MacCary (1971), and Taylor, Hess, and Wheatcraft (1990).

b. Planning a well logging program. The objective of any well logging program should be to acquire data on a real-time basis and to develop the background data to be able to monitor changes in the borehole environment over time. Borehole geophysical logs require calibration in the geologic environment in which they will be run. This is because logs have non-unique response, and there are no published or standard correction factors for many geologic media common to groundwater studies, such as all igneous rocks, metamorphic rocks, and certain sedimentary rocks such as conglomerates. Calibrating for a geologic environment in which little or no data is available will require that core samples be obtained and tested for physical properties such as density, porosity, and saturation. Logging company contracts typically contain a clause which states that they are not responsible for the quality of the data. This principle is part of the larger concept of a quality assurance/ quality control (QA/QC) program. At a minimum, a logging QA/QC program should consider the following:

(1) Calibration. When was the tool last calibrated and how? Tools should be calibrated at standard pits on a regular basis. These pits are located at the University of Houston and at the U.S. Geological Survey Denver Field Center. Calibration also means the use of field standards to check the tool at the beginning and end of each day.

(2) Core analysis. Preferred, necessary in new area.

(3) Water analyses. Essential, also includes mud analyses.

(4) Well construction details. Essential if logging inside existing well.

(5) Local hydrogeology. Essential for understanding logs and anticipating problems and/or anomalies.

(6) Logging procedures. Essential, requires onsite presence. An example is logging speed. Some logs should run at speeds as low as 7.5 m/min (25 ft/min).

(7) Data processing. All logs need some correction. Depth is commonly ±5 m (15 ft), scales off up to ±20 units. Additionally, borehole effects need to be corrected for. In order to perform this type of error analysis, the data (log) must be digitized so corrections can be made and the data replotted.

(8) Drilling. Carry out drilling operations in a manner that produces the most uniform hole and least disturbance to the formation.

The final principle to keep in mind is that logs should always be interpreted collectively, on the basis of a thorough understanding of the principles and limitations of each type of log, and a basic understanding of the hydrogeology of the study area. Table 4-1 summarizes the application of various types of logs. Figure 4-3 presents an example of the conjunctive use of borehole geophysical logs.

c. Types of logs.

(1) Caliper log.

(a) Principle. A caliper log is a record of the average borehole diameter. It is one of the first logs

Table 4-1
Applications of various borehole geophysical methods

Parameter	Borehole Geophysical Method
Stratigraphy and porosity	Natural gamma log Gamma-gamma log Acoustic log Neutron log Spontaneous potential log
Stratigraphy	Caliper log Resistance log
Moisture content	Neutron log
Location of zones of saturation	Spontaneous potential log Temperature log Neutron log Gamma-gamma log
Physical and chemical characteristics of fluids	Resistivity log Spontaneous potential log Temperature log Fluid conductivity log
Dispersion, dilution, and movement of waste	Fluid conductivity log Temperature log Gamma-gamma log

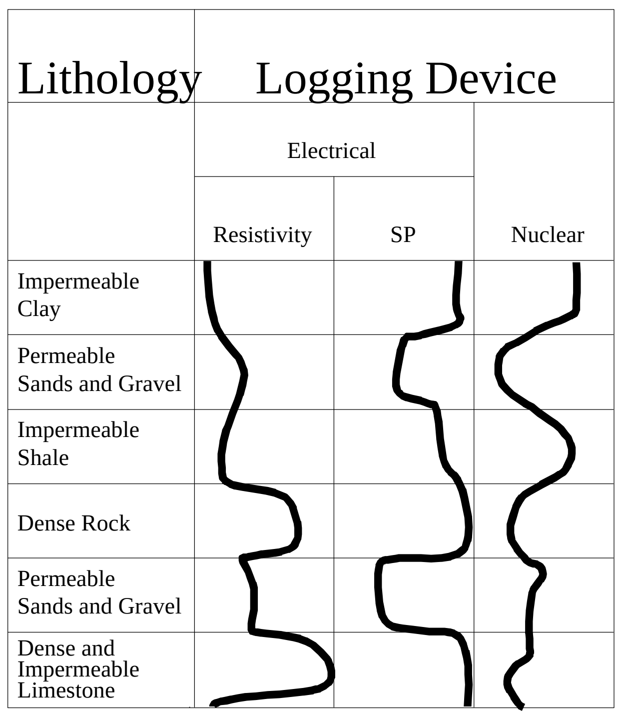

Figure 4-3. Conjunctive use of borehole geophysical logs

which should be run, and one of the most useful and simplest tools. The caliper log is necessary for the selection of the size of other tools and for any borehole corrections to other logs. Most caliper tools consist of a body or sonde with from one to four "arms" that follow the borehole wall.

(b) Application. Caliper logs are utilized for identification of lithologic horizons, location of fractures or other openings in the borehole wall, guidance in well design and construction, and most importantly, for the borehole correction needed for other logs such as single point resistance and gamma-gamma.

(2) Fluid conductivity and temperature logs.

(a) Principle. Fluid-conductivity logs provide a measurement of the conductivity of the fluid within the borehole, which may or may not be related to the fluid(s) in the formation. Generally, the conductivity is derived from measuring the potential drop between two closely spaced electrodes.

(b) Application. Fluid-conductivity measurements are needed for the correction of other logs which are sensitive to changes in the electrochemical nature of the borehole fluid(s) such as spontaneous potential and most resistance logs. It is good practice to make a temperature log simultaneously with the fluid-conductivity log. This allows the most accurate conversion to specific conductance, which is needed to calculate equivalent salinity, by avoiding the disturbance of the fluid column that may be induced by running a separate thermal probe prior to the fluid-conductivity probe. Fluid conductivity and temperature logs should be run at the beginning of the logging process.

(3) Spontaneous potential log.

(a) Principle. The spontaneous potential (SP) tool measures the natural electric potential between borehole fluid and the formation. The SP log must be run in an uncased borehole filled with a conductive fluid. Two sources of potential are recognized. The first, and least significant, is the streaming potential caused by dissolved electrolytes such as NaCl, moving through a porous media. This phenomenon occurs where filtrate is lost to the formation or where the borehole is gaining or losing fluid. The second, and most significant, is the electrochemical reaction that occurs at the interface between dissimilar materials. Currents tend to flow from the borehole into permeable beds until sufficient cross-sectional area of a resistant bed is encountered to carry the current.

(b) Application. The SP log can be used for lithologic identification or correlation. As previously discussed, the direction of the deflection of the SP log is an indication of either sand or shale. It is only a qualitative indicator and should never be used alone. Thus, SP should not be used for calculating water quality during hydrogeologic investigations.

(4) Resistance logging.

(a) Principle. Resistance logging provides a calculation of the resistance, in ohms, of the geologic materials between an electrode placed within the borehole and an electrode placed at the surface or between two electrodes placed within the borehole. The resistance log must be run in an uncased borehole filled with a conductive fluid. A potential difference in volts or millivolts is measured between the two electrodes and the resistance is calculated by Ohm's Law when the current I is held constant:

$$E = I\,r \qquad (4\text{-}1)$$

where

E = potential [volts]

I = current [amperes]

r = resistance [ohms]

(b) Application. The most common type of resistance logging is the single point or point resistance method. Because the radius of investigation is small, the single point resistance log is strongly affected by the conductivity of the borehole fluid and variation of borehole diameter. Single point resistance logging is very useful because any increase in formation resistance produces a corresponding increase in the resistance recorded on the log and thus deflections on a single point log can be attributed to changes in lithology (although the response is nonlinear). Single point systems do not experience the log reversal at thin beds that multielectrode systems do. These properties make the single point log one of the better logs for lithology or stratigraphic correlation.

(5) Resistivity logging.

(a) Principle. Though their names are similar, resistivity logging is different from resistance logging. Resistivity includes the dimensions of the material being measured and is an electrical property inherent to the geologic material. The relationship between resistance and resistivity is analogous to that of weight and density. Resistivity is defined by:

$$R = r \times S / L \tag{4-2}$$

where

R = resistivity [ohm-meters]

r = resistance [ohms]

S = cross-sectional area [L^2]

L = length [L]

The resistivity of sediments depends on the physical properties of those sediments and the fluid(s) they contain. Most sediments are composed of particles of very high electrical resistance. All resistivity logs must be run in an uncased borehole filled with a conductive fluid. When saturated, the water filling the pore spaces is relatively conductive compared to the sediments. Thus, the resistivity of sediments below the water table is a function of the salinity of the water filling pore spaces and how those pore spaces are interconnected. Resistivity logging devices measure the electrical resistivity of a known (assumed) volume of geologic material using either direct or induced electrical currents. Below the water table the resistance of a formation depends on the composition of the water within it, and on the length and shape of interconnected pores.

(b) Application. Resistivity logs are generally used to estimate the physical and chemical characteristics of fluids, formation resistivity and porosity, and mud resistivity. Normal logs are the most commonly used resistivity tools in groundwater investigations. Normal logs measure the apparent resistivity of a volume of geologic material surrounding the electrodes. The records produced by normal devices are affected by bed thickness as well as bed resistivity. As bed thickness decreases, the resistivity peak decreases in amplitude.

(6) Natural-gamma logs.

(a) Principle. Some of the most useful logging methods involve the measurement of either natural radioactivity of the geologic media and the fluids within it, or the attenuation of induced radiation. Nuclear methods can be used in either open or cased boreholes provided there are not multiple casing strings and seals. Natural-gamma logs are records of the amount of natural-gamma radiation emitted by geologic materials. The chief uses of natural-gamma logs are the identification of lithology and stratigraphic correlation. Potassium, of which about 0.012 percent is K_{40}, is abundant in feldspars and micas which decompose readily to clay. Clays also concentrate the heavy radioelements through the processes of ion exchange and adsorption. In general, the natural-gamma activity of clay-rich sediments is much higher than that of quartz sands and carbonates. The radius of investigation of a natural-gamma probe is a function of the probe, borehole fluid, borehole diameter, size and number of casing strings and seals, density of the geologic materials, and photon energy.

(b) Application. The most important application in groundwater studies of the natural-gamma log is the identification of clay- or shale-bearing sediments. Clays tend to reduce the effective porosity and hydraulic conductivity of aquifers, and the natural-gamma log can be used to empirically determine the shale or clay content in some sediments. The natural-gamma log does not have a unique response to lithology. The response is generally consistent for a given locality.

(7) Gamma-gamma logs.

(a) Principle. Gamma-gamma logs are records of the intensity of gamma radiation from a source in the probe after the radiation has been backscattered and attenuated within the borehole and surrounding geologic materials. The gamma-gamma probe contains a source of gamma radiation, generally cobalt-60 or cesium-137, shielded from a sodium iodide detector. Gamma radiation from the source penetrates and is scattered and absorbed by the fluid, casing, grout, and geologic materials surrounding the probe. Gamma radiation is absorbed and/or scattered by all material through which it travels. The radius of investigation of gamma-gamma probes is reported to be about 15 cm (6 in.). The bulk density of the geologic material, fluids, or casing and seals will affect the radius of investigation. The distance between the source and detector will also significantly alter the volume of material investigated.

(b) Application. The main uses of gamma-gamma logs are for the identification of lithology, and the measurement of bulk density and subsequent determination of porosity. They may also be used to locate cavities and grout outside of casing. Gamma-gamma density is widely used for determination of total porosity by:

$$\text{Porosity} = \frac{\text{Grain density} - \text{Bulk density (from log)}}{\text{Grain density} - \text{Fluid density}} \tag{4-3}$$

Grain density can be derived from laboratory analyses of cores (or, for quartz sands, a value of 2.65 g/cc can be used). The fluid density for groundwater studies is assumed to be 1 g/cc; however, if the fluid is saline or contains levels of contaminants high enough to alter fluid density, then laboratory analysis of density is necessary. In an unconfined aquifer or a partially dewatered confined aquifer, it should be possible to derive specific yield from gamma-gamma logs. Specific yield should be proportional to the difference between the bulk density of saturated and drained sediments, assuming porosity and grain density do not change. Bulk density may be read directly from a calibrated and corrected log or derived from charts providing correction factors. Errors in bulk density obtained by gamma-gamma methods are on the order of ± 2 percent. Errors in the porosity calculated from log-derived bulk densities depend upon the accuracy of grain and fluid densities used. In addition to determining porosity, gamma-gamma may be used to locate casing, collars, or the position of grout outside the casing. Gamma-gamma logs can also indicate water level and significant changes in fluid density (fresh water-brine interface). A license must be obtained to use a gamma-gamma log.

(8) Neutron logs.

(a) Principle. The various types of neutron logs are potentially the most useful techniques in borehole geophysics as applied to groundwater studies. This is due to the fact that the measured response is due to hydrogen and thus, generally, water. Neutron logs also have advantages over other nuclear logs in that they can be run in liquid-filled or dry holes, cased or uncased holes, and have a relatively large volume of influence. In neutron logging, neutrons are artificially introduced into the borehole environment, and the effect of the environment on the neutrons is measured. Assuming that the vast majority of hydrogen occurs in the form H_2O, materials with higher porosity (and thus higher water content) will slow and capture more neutrons, resulting in fewer neutrons reaching the detector. The converse is true for materials of low porosity. This assumption does not hold when hydrocarbons, chemically or physically bound water, and/or other hydrogenous materials are present. Neutron logs are affected by changes in borehole conditions to a lesser degree than other geophysical logs that measure the properties of geologic materials. The most marked extraneous effect on neutron logs is caused by changes in borehole diameter.

The volume of influence, which is defined by the radius of investigation, is an important factor in the analysis of neutron logs. The radius of investigation is a function both of the source-detector spacing, the petrology of the material, and the water content within the volume of influence. The radius of investigation of neutron tools has been reported to be from 15 cm (6 in.) for high-porosity saturated materials to 60 cm (2 ft) in low-porosity materials. These estimates may be conservative, as recent laboratory work with semi-infinite models suggests that the radius of investigation in saturated sands was greater than 50 cm (20 in.).

(b) Application. Neutron logs are used chiefly for the measurement of moisture content in the unsaturated zone and total porosity (water filled) in the saturated zone. In most geologic media the hydrogen content is directly proportional to the interstitial-water content; however, hydrocarbons, chemically or physically bound water, or any hydrogenous material can give anomalous values. For example, gypsum has a high percentage of water associated with the crystal structure which can result in it appearing to be a material of high porosity. This has been used to distinguish between anhydrite (high neutron count rate) and gypsum (low neutron count rate). Although a neutron log cannot be used to measure porosity above the water table, it is very useful for measuring changes of water content in the unsaturated zone. A license is required to use a neutron log.

(9) Acoustic logs.

(a) Principle. Acoustic logging utilizes a transducer to transmit an acoustic wave through the borehole fluid and into the surrounding rocks. The four most common types of acoustic logs are: acoustic velocity, acoustic wave form, cement bond, and acoustic televiewer. Acoustic logs can provide data on porosity, lithology, cementation, and fractures. Acoustic logging is appropriate only for consolidated (cemented) material. Acoustic-velocity logs, also called sonic logs or travel-time logs, are a record of the travel time of an acoustic wave from one or more transmitters to receivers in a probe. The velocity of the acoustic signal is related to the mineralogy and porosity of the formation. The radius of investigation of an acoustic-velocity probe is reported to be approximately three times its wavelength; the wavelength is equal to the velocity divided by the frequency. At a frequency of 20 kHz, this radius ranges from less than 30 cm (1 ft) in unconsolidated materials to about 120 cm (4 ft) in hard rocks.

(b) Application. Acoustic-velocity logs are useful for providing information about lithology and porosity when used in consolidated materials penetrated by uncased, fluid-filled boreholes. Transit times decrease with greater depth and with increases in cementation. Acoustic velocities may vary with confining pressure for several hundred feet below the ground surface, most notably in slightly consolidated materials. Secondary porosity will not be detected by an acoustic-velocity log because the acoustic wave will take the fastest path through the formation. Intervals of secondary porosity can be identified by cross-plotting data from an acoustic-velocity log and a neutron log or a gamma-gamma log.

4-9. Surface Geophysics

a. General. Surface geophysical methods generally do not provide the vertical resolution of borehole geophysical methods. However, surface geophysical methods provide valuable information on site geology on a greater spatial scale. Thus, conceptual model development often requires the conjunctive use of surface and borehole geophysical methods. Additionally, surface geophysical methods allow for the nonintrusive gathering of information on subsurface stratigraphy and hydrogeologic conditions. This section presents an overview of: seismic refraction and reflection, electrical resistivity, gravitational methods, electromagnetic methods, and ground-penetrating radar. Recommended references are included when a more in-depth understanding of concepts and principles is desired.

b. Seismic geophysical methods.

(1) The seismic exploration method deals with the measurement of the transmission, refraction, reflection, and attenuation of artificially generated seismic waves traveling through subsurface materials. The refraction and reflection methods are the most widely used seismic methods for hydrogeologic site characterizations. Both of these methods make use of the fact that seismic waves travel through different materials, such as soil, weathered rock, intact rock, etc., with differing velocities. Measurements are made by generating a seismic disturbance at or just below the ground surface and measuring the time required for the disturbance to travel through the ground and to one or more seismic sensors, called geophones, which are firmly implanted into the ground surface. With a suitable geometric arrangement of the seismic source and geophones, and theory to determine the probable travel paths, considerable information can be gained on the geometry and stratigraphy of the underlying soil and rock materials. In some cases, particularly in unconsolidated sediments, the depth to the water table may be computed.

(2) This section provides a brief overview of the seismic refraction and reflection methods, along with applications in hydrologic site characterization studies. Also, the strengths and weaknesses of each method will be assessed. For a more in-depth discussion, the following references are recommended: EM 1110-1-1802; Telford et al. (1990); and Zohdy, Eaton, and Mabey (1974).

c. Types of seismic geophysical methods.

(1) Seismic refraction.

(a) Principle. Seismic refraction technology is based on the fact that elastic waves travel through differing earth materials at different velocities. The denser the material, the higher the wave velocity. When seismic waves are propagated through a geologic boundary of two layers with separate densities, a refracting of propagation direction occurs. Through the propagation of a set of elastic waves, usually through small explosions, and the recording of the time travel at differing distances on a seismograph, the layer depths and their acoustic velocities can be estimated. Seismic refraction methods are only effective in formations with definite boundaries between strata and where density increases with each successive lower layer.

(b) Application. The acoustic velocity of a medium saturated with water is greatly increased in comparison with velocities in the vadose zone. Thus, the refraction method is applicable in determining the depth to the water table in unconsolidated sediments. The velocities associated with those of saturated unconsolidated materials, although indicative of saturation, are by no means unique. For example, a dry, weathered rock layer can exhibit the same velocity ranges as those normally associated with saturated, unconsolidated materials. The refraction method is also applicable in determining the depth and extent of a rock aquifer as well as the thickness of overlying unconsolidated materials. Common hydrologic problems that relate to this situation are that of mapping buried channels or determining the thickness of unconsolidated materials, whether saturated or not, in a bedrock valley.

One of the major limitations of the seismic refraction method is that each successive velocity layer must have a velocity greater than the one above it. If a low-velocity layer is between layers with greater velocities, the low-velocity stratum will not be detected and erroneous depths to deeper interfaces will be computed. However, in most hydrologic cases an increase in velocity as a function of depth can be expected (such as the case when there is a water table in sediments which are underlain by bedrock). Another limitation of the method is its inability to detect thin intermediate velocity layers. An example of this situation is a relatively thin saturated zone at the bottom of a thick sand layer which overlies a high-velocity bedrock surface. In this case, the refracted arrivals from the bedrock arrive prior to those from the top of the saturated sand and, as a consequence, the saturated layer will not be detected.

(2) Seismic reflection.

(a) Principle. The basic principle of seismic reflection is that seismic waves are reflected at interfaces between geologic units with different seismic velocities. Seismic velocities depend upon the elastic constants of a porous medium. The time required for an acoustic signal to travel from the source to the reflecting stratigraphic boundary and back to a defined point on the surface is measured by a geophone (Figure 4-4). The geophone detects the reflected signals from the various reflecting horizons and transmits this information to a seismograph where the times of arrival are recorded. By measuring the time the energy takes to propagate from the source to a reflecting horizon and back to the surface and by also knowing the velocity of the material along the path of travel, the depth to the reflecting horizons can be computed. By recording the output of each geophone in a seismic line, a visual representation of the local pattern of ground motion called a seismogram is obtained (Figure 4-5). By displaying the seismogram for each geophone side by side, a vertical profile of the subsurface may be obtained.

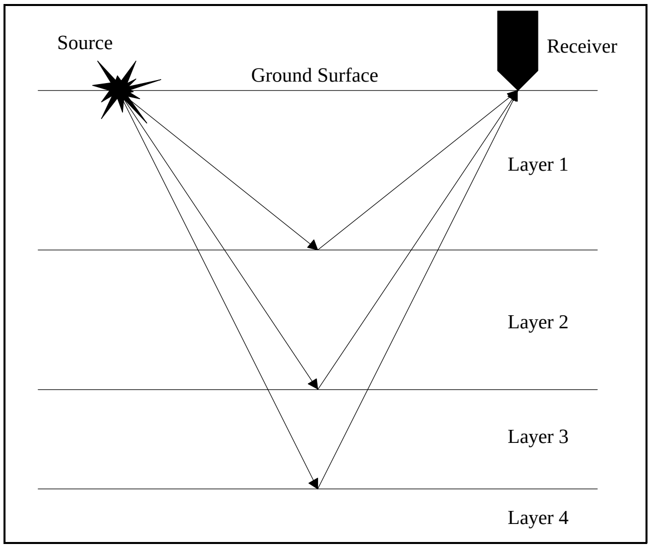

Figure 4-4. Reflected energy from three layers

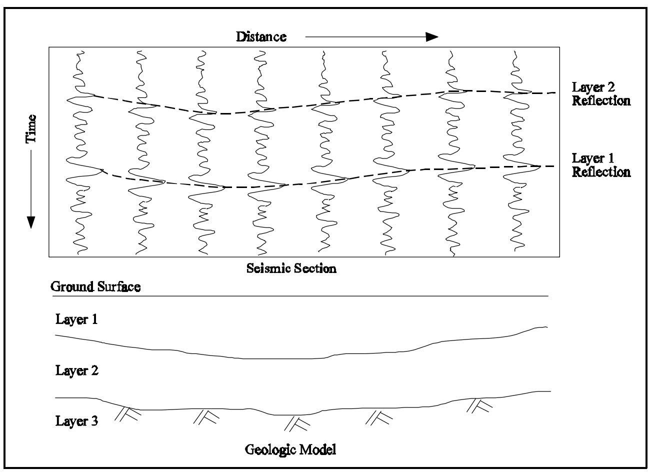

Figure 4-5. Illustration of seismic section for a hypothetical geologic model

(b) Application. One important advantage the reflection method has over the refraction method is that succeeding layers do not have to increase with velocity. The reflection method is more accurate, has better resolution, and can determine the depths to more layers than the refraction method. It also uses shorter geophone lines and uses smaller energy sources. Seismic reflection methods are capable of producing detailed information on subsurface structure. However, interpretation of seismic reflection data requires analysis by a trained geophysicist using powerful computer techniques; moreover, the collection of data requires expensive field equipment and large field teams (Smith and Wheatcraft 1991).

d. Additional surface geophysical methods.

(1) Surface electrical resistivity.

(a) Principle. Resistivity of a material is defined as the ability of that material to impede the flow of electric current through the material. In a surface resistivity survey, a direct current or low-frequency alternating current is sent through the ground between metal stakes or electrodes. The accompanying drop in electric potential is measured at a point between the current electrodes. Electrical resistivity displays a wider range of values than any other physical property in rocks (Zohdy, Eaton, and Mabey1974). Resistivity depends primarily on the amount, distribution, and salinity of water in the rock being studied. Saturated rocks have lower resistivities than unsaturated and dry rocks. Electrical resistivity methods are most useful in determining depth to rock and evaluating stratified formations where a denser stratum overlies a less dense stratum. Clays and conductive materials also reduce the rock's resistivity. An in-depth discussion of surface electrical resistivity can be found in EM 1110-1-1802.

(b) Application. Electrical resistivity methods have a variety of applications. Although the field techniques are relatively time-consuming, it is often the chosen method because surface resistivity surveying is one of the less costly geophysical techniques. Resistivity surveying is commonly used in groundwater studies for determining the water table depth, locating freshwater aquifers, mapping confining clay layers, and mapping saltwater intrusion and contaminant plumes. Other applications of electrical resistivity include: determination of depth to bedrock, cavern location in karst regions, permafrost mapping, and geothermal exploration.

(2) Gravitational methods.

(a) Principle. Gravitational methods are based on measurement of small variations in the gravitational field at ground surface. If subsurface rocks of differing density are present in the study area, the resulting irregularity in mass distribution will be reflected in a corresponding change in gravity intensity on the surface. Geophysical gravity surveys involve measurement of the magnitude and spatial variation of the earth's gravity field using a gravity meter. If the earth were perfectly spherical and radially uniform, the acceleration of gravity would be constant over the earth; however, this is not the case. Measurement, definition, and geological interpretation of the departures from radial uniformity are the objectives of gravity surveys; some of the departures are global or regional in scale, while some of the departures are local in nature. Local departures from radial uniformity in the subsurface are referred to as anomalies and give rise to local gravity variations or gravity anomalies. The concept of a gravity anomaly is illustrated in Figure 4-6, where the terms total or measured gravity, regional gravity trend, residual gravity, and gravity anomaly are defined.

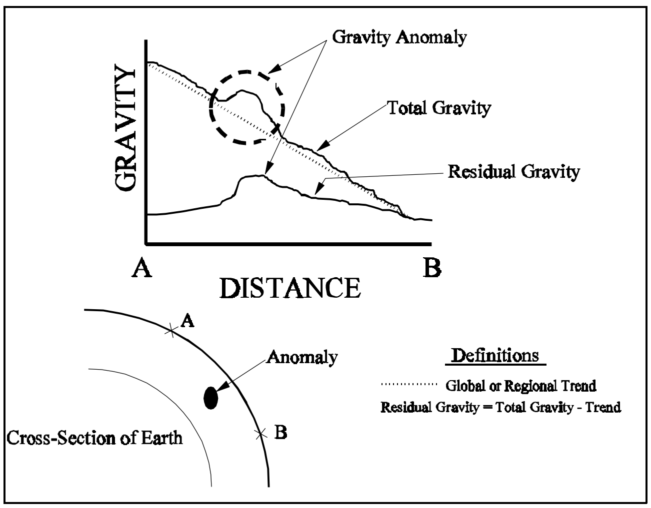

Figure 4-6. Concept of gravity anomaly and definition of residual gravity anomaly

(b) Application. For hydrogeologic investigations, gravity surveys have two primary applications. The first of these applications contributes to the definition of local- to regional-scale geology and is a standard or classical use of gravity surveys. Gravity surveys can be used to map “bedrock” topography, detect and map buried river channels (Figure 4-7), detect and map large fracture zones, and detect truncations or “pinchouts” of major aquifers or aquitards. The second application area for gravity surveys in hydrogeologic investigations involves the determination of fundamental hydrogeologic parameters or properties. Gravity surveys can be used for monitoring gravity changes associated with groundwater level changes; if the bulk porosity is known, the elevation change can be determined from the gravity change, or if the elevation change is known from a monitor well, a representative bulk porosity can be determined from the gravity change. An emerging application area for gravity surveys is to monitor gravity changes associated with a pumping well. Theoretically, if gravity surveys are conducted before and during well pumping, the shape of the drawdown curve can be determined, flow heterogeneity can be mapped, and estimates of bulk hydraulic conductivity can be determined from the gravity data. Telford et al. (1990) and Carmichael and Henry (1977) discuss standard gravity survey procedures in detail, and Butler (1980) discusses procedures for microgravity surveys.

(3) Electromagnetic methods.

(a) Principle. The electromagnetic (EM) method involves the propagation of time-varying, low-frequency electromagnetic fields in and over the earth. The electromagnetic method provides a means of measuring the electrical conductivity of subsurface soil, rock, and groundwater. Electrical conductivity is a function of the type of soil and rock, its porosity, its permeability, and the fluids which fill the pore space. In most cases the conductivity (specific conductance) of the pore fluids will dominate the measurement. Accordingly, the electromagnetic method is applicable both to assessment of natural hydrogeologic conditions and to mapping of many types of contaminant plumes. Additionally, trench boundaries, buried wastes and drums, as well as metallic utility lines can be located with electromagnetic techniques.

Natural variations in subsurface conductivity may be caused by changes in soil moisture content, groundwater specific conductance, depth of soil cover over rock, and thickness of soil and rock layers. Changes in basic soil or rock types, and structural features such as fractures or voids may also produce changes in conductivity. Localized deposits of natural organics, clay, sand, gravel, or salt-rich zones will also affect subsurface conductivity.

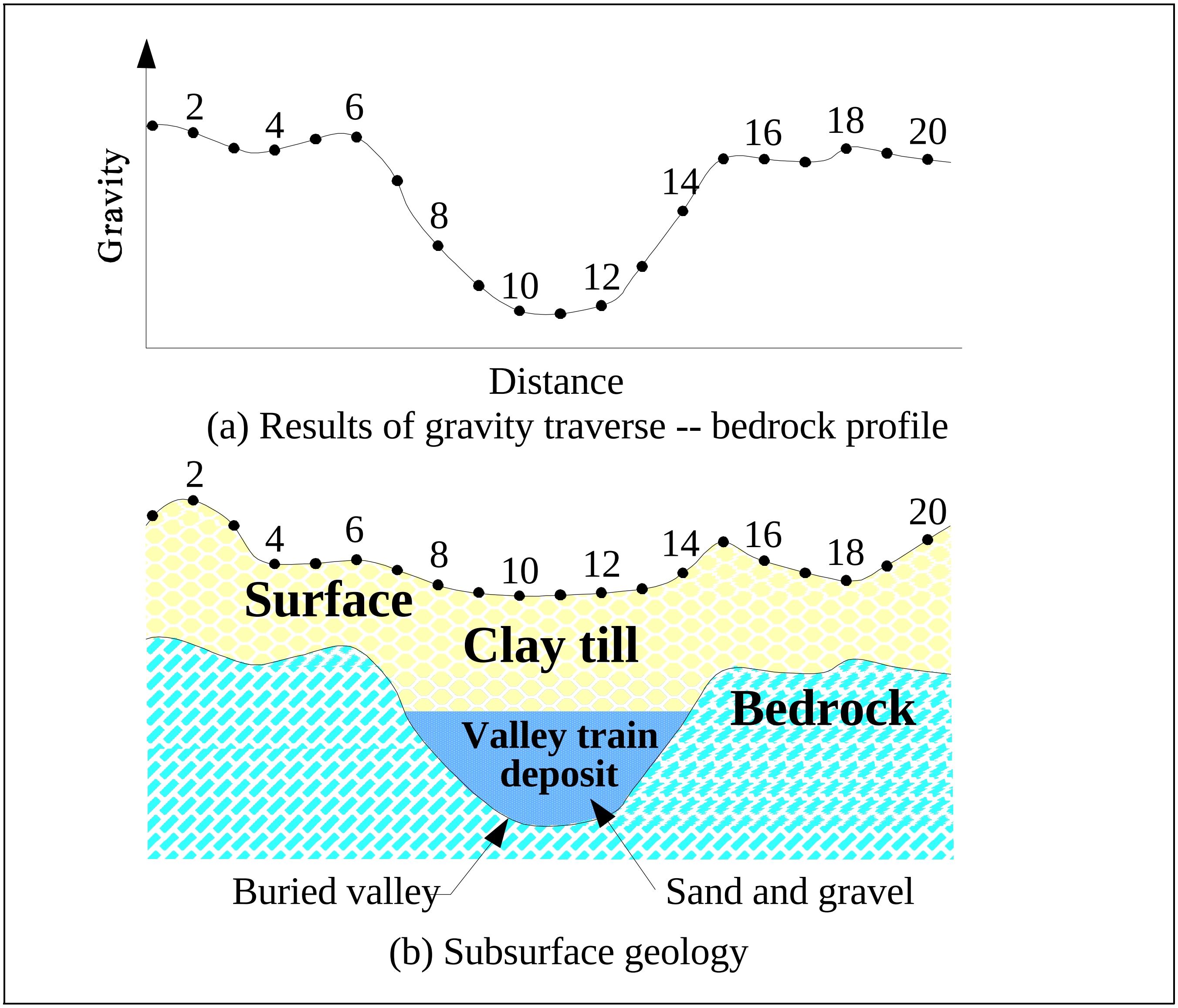

Figure 4-7. Illustration of Gravity Anomaly above a Buried Channel

(b) Application. There are two basic techniques available for electromagnetic surveying. Profiling is accomplished by making fixed-depth electromagnetic measurements along a transverse line to detect lateral variations. Sounding is accomplished by making conductivity measurements to various depths at a given location to detect vertical variations.

Electromagnetic systems are susceptible to signal interference from a variety of sources, originating both above the ground and below. Electromagnetic noise may be caused by nearby power lines, powerful radio transmitters, and atmospheric conditions. In addition to other forms of electromagnetic noise, instrument responses from subsurface or surface metal may make it difficult to obtain a valid measurement. Unique interpretation of subsurface conditions generally cannot be obtained from electromagnetic sounding data alone; it must be supported by drilling data or other geologic information.

(4) Ground-penetrating radar theory.

(a) Principle. Ground-penetrating radar (GPR) utilizes high frequencies of electromagnetic waves which are propagated in a straight line into the ground to depths which vary from a few feet to tens of feet, depending on the electrical conductivity of the terrain. The use of GPR is similar to the seismic reflection technique because both methods record the time required for a wave to travel to an interface between two formations and then reflect to the surface. In general, electromagnetic methods lack the resolution and depth penetration of resistivity surveys, but have the advantage of being rapid and less expensive.

In geologic materials, the presence of water is one of the most important factors determining electrical properties. In addition, ions dissolved in the water give rise to an electrical conduction mechanism which is a major factor in most soils and rock. Basically the conductivity is roughly proportional to the total dissolved solid content; hence, the more ions dissolved in the solution, the higher the conductivity. The electrical conductivity of a soil is much harder to predict. It is very dependent on the pore-water conductivity. In addition, it is dependent on the surface conduction mechanisms present in the soil matrix. Surface conduction addresses the charge transport associated with charges moving on the surface of the mineral grains. Generally surface conduction is very small in clean, coarse-grained material such as quartz sand; however, it is a major factor in fine-grained soils such as clays. As a result, clays are very important in GPR investigations because they have a strong impact on electrical conductivity of the medium. As electromagnetic waves propagate downward into the ground, reflections are generated by changes in the electrical impedance in the ground.

(b) Application. Before starting a GPR survey, one must determine if the site conditions and desired target are suitable. Of primary concern is the target depth; GPR has a very definite and often limited depth of investigation based on the site geology. Clay and saturated soils attenuate the GPR signal, thereby severely limiting the depth of penetration. The target size should also be qualified as accurately as possible. In order for GPR to work, the target must present a contrast in electrical properties to the host environment in order that the electromagnetic signal be modified, reflected, or scattered. The host material must be qualified in two ways. First, the electrical properties of the host must be defined. Second, the degree and spatial scale of heterogeneity in the electrical properties of the host must be estimated. If the host material exhibits variations in properties which are similar to the contrast and scale of the target, the target may not be recognizable from the responses generated by the host environment. Lastly, the area where the survey is to be performed should be free of the presence of extensive metal structures and of radio frequency electromagnetic sources or transmitters.

4-10. Cone Penetrometer Testing

a. General. Cone penetrometer testing (CPT) has been utilized in the geotechnical field for at least 65 years. Benefits of using the CPT system include lower costs, faster data acquisition, less invasive disturbance to the subsurface, and no acquisition-derived wastes. A CPT apparatus is typically truck-mounted, similar to a drilling machine. A basic CPT system consists of four basic components; the truck, hydraulic thrust system, data acquisition and reduction system (computers), and the sensor assembly, i.e. the cone. (Figure 4-8). The truck not only transports the CPT unit, but also supplies the power to drive the CPT system. In addition, the truck also provides the mass necessary to counteract the hydraulic thrust. Truck sizes vary anywhere from 4,400 to 28,500 kg, with 17,500 kg being most common.

b. Hydraulic thrust system. The hydraulic thrust system provides the force to push the sounding rod(s)/cone assembly into the ground. The depth of penetration is a function of several factors; truck weight, soil density and cementation skin friction on rods, and deviation (from vertical) of rods. Depending upon the interaction of these factors, CPT has been done at depths up to 100 m (300 ft), with depths of 25 to 30 m (80 to 100 ft) routine. This makes CPT a truly practical alternative to drilling.

c. Data acquisition system. The data acquisition and reduction system receives the signals from the sensor in the cone assembly and processes them,

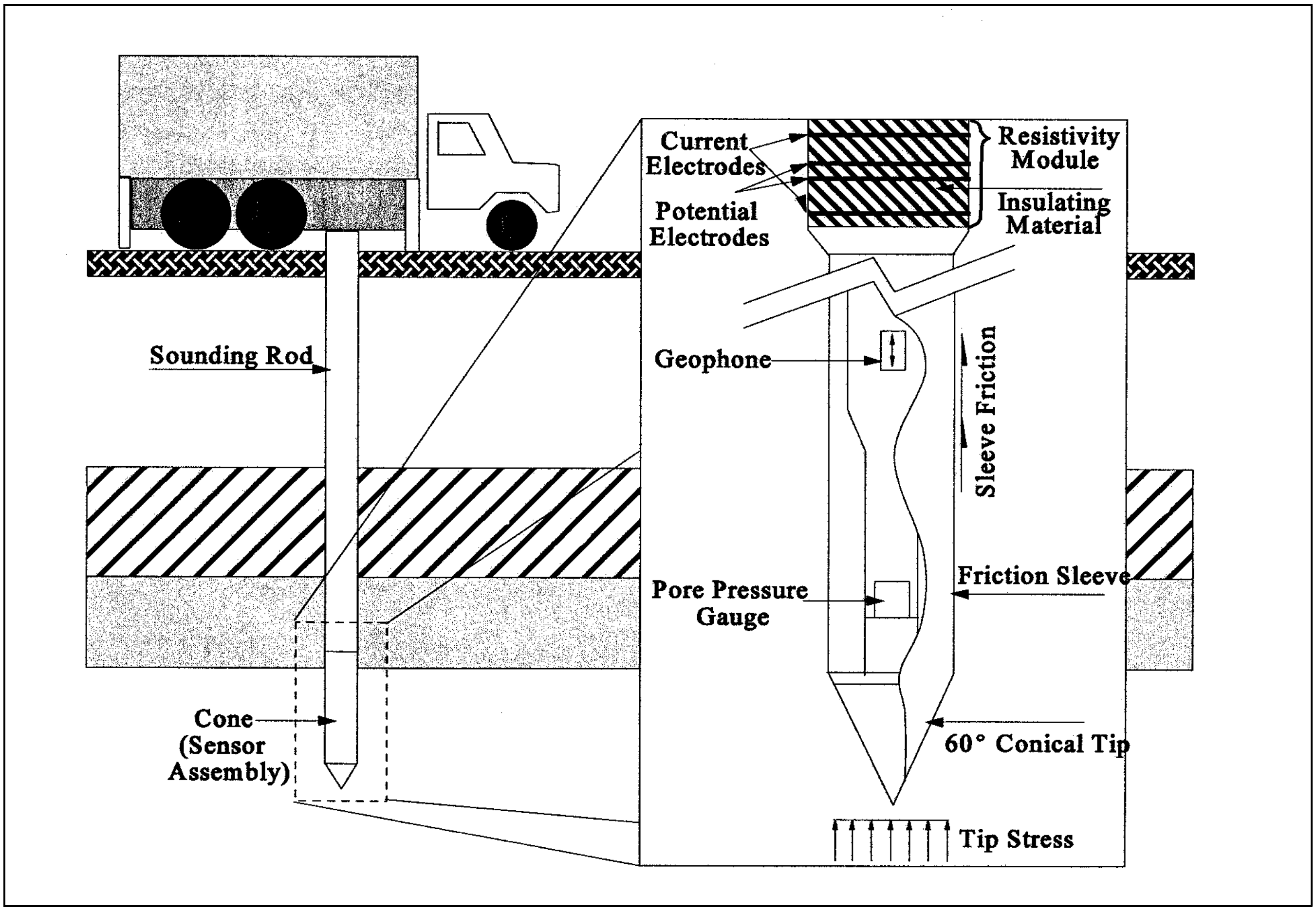

Figure 4-8. Components of cone penetrometer testing system

providing both digital and hard copy of raw data and basic interpretation. Interpretations are in the form of soil behavior types, which are based upon a database relating the ratio of skin friction to the tip resistance to soil type. In addition, the data acquisition and reduction system performs system monitoring to ensure the sensors are functioning properly. The type of data that can be acquired and processed is limited only by the selection of sensors. Currently, there are proven sensors for soil type (stratigraphy), pore pressure (water content/water table), soil electrical resistance, seismic velocity, radiation (gamma), laser-induced florescence, temperature, pH, and soil gas and groundwater sampling. The CPT system does not produce samples for direct observation. Thus, it is preferable for the CPT to be used in conjunction with lithologic data obtained from standard drilling methods.

d. Sensor assembly. The cone assembly houses all the sensor elements. The sensors within the cone are connected to the data acquisition and reduction system by a multi-lead electrical cable which runs through the center of the sounding rods. When sampling soil, groundwater, or soil gas, the cone containing the sensor elements fig4-8is replaced with a “dummy.” The dummy has a tip which is pushed ahead of the sounding rods, exposing the annulus of the rods to the environment, allowing the insertion/use of various soil, water, and gas samplers.

e. Limitations. Limitations of the CPT method include:

(1) Smaller trucks require an anchoring system which is sometimes difficult to get.

(2) Large trucks with reaction mass pose access problems.

(3) Rocks/debris in the near surface can interfere with data acquisition.

f. Site characterization and analysis penetrometer system (SCAPS). SCAPS is a resource to be used by all Corps districts and laboratories for the investigation of HTRW sites. An in-depth explanation of SCAPS capability, along with points of contact, can be found in ETL 1110-1-171.

4-11. Isotope Hydrology

a. General. Isotopes are atoms of the same element that have different masses; they have the same number of protons and electrons, but a different number of neutrons (Figure 4-9). The 92 natural elements give rise to more than 1,000 stable and radioactive isotopes. These are often called environmental isotopes. Environmental isotopes are commonly categorized into two general groups: stable isotopes and unstable isotopes. Stable isotopes are not involved in radioactive decay. Most stable isotopes do not react chemically in the subsurface environment and are of particular use in determining the source of groundwater. Unstable isotopes are undergoing decay. Unstable isotopes are of particular use in determining the age of water. The relative abundance of isotopes of hydrogen, oxygen, and carbon in the hydrologic cycle is presented as Table 4-2. An in-depth discussion on the use of environmental isotopes can be found in Fritz and Fontes (1980).

Mass Number

$^{12}_{6}C$ 6 protons, 6 electrons, 6 neutrons

Atomic Number

$^{14}_{6}C$ 6 protons, 6 electrons, 8 neutrons (Isotope)

Figure 4-9. Example of carbon isotope ^{14}C

Table 4-2
Relative Abundance of Isotopes of Hydrogen, Oxygen, and Carbon in the Hydrologic Cycle (Freeze and Cherry 1979)

Atom	Relative Abundance (%)	Type
^{1}H	99.984	
^{2}H (deuterium)	0.016	Stable isotope
^{3}H (tritium)	$0\text{-}10^{-15}$	Radioactive isotope (½ life 12.3 yr)
^{16}O	99.76	
^{17}O (oxygen-17)	0.04	Stable isotope
^{18}O (oxygen-18)	0.20	Stable isotope
^{14}C (radiocarbon)	<0.001	Radioactive isotope (½ life 5,730 yr)

b. Stable isotopes.

(1) General. Stable isotopes can serve as natural tracers that move at the average velocity of groundwater, and are of particular use in determining the recharge areas, degrees of mixing between waters of different origin, and hydrograph separation. The most common stable isotopes used in hydrologic analysis are ^{18}O and ^{2}H (also known as deuterium or D).

(2) Isotopic fractionation. In lighter elements, such as hydrogen (H), and oxygen (O), the differences in mass produced by isotopes is significant to the total mass of the atom. These differences in mass cause isotopic fractionation in nature. Isotopic fractionation is any process that causes the isotopic ratios in particular phases or regions to differ from one other. For example, the ratio of $^{16}O/^{18}O$ in rain is different from the ratio in the oceans. This ratio is represented by δ:

$$\delta^{18}O = \frac{(^{18}O/^{16}O)_{sample} - (^{18}O/^{16}O)_{standard}}{(^{18}O/^{16}O)_{standard}} \times 1{,}000 \qquad (4\text{-}4)$$

Similarly, the isotopic fractionation ratio of ^{2}H (deuterium-D) is represented by δD. For a specific area, the relationship between $\delta^{18}O$ and δD in rainfall is approximately linear and can be plotted on a meteoric water line, an empirically derived relationship for continental precipitation.

(3) Use of stable isotopes as natural tracers. Isotopic fractionation of $\delta^{18}O$ and δD during phase changes enriches one isotope relative to another. Water that has been subject to evaporation is enriched in ^{2}H (D) relative to ^{18}O because of its lower atomic weight. Fractionation is temperature-dependent. For example, winter precipitation is depleted in ^{18}O and ^{2}H compared with summer precipitation. Additionally, precipitation at the beginning of a storm is often higher in ^{18}O and ^{2}H than at the end of the storm, as the heavier isotopes are selectively removed from the vapor phase. These processes provide water masses with unique signatures that can be used as natural tracers in groundwater studies. Hence, the source areas of different waters, and the mixing patterns between waters can be assessed.

(4) Use of stable isotopes for hydrograph separation. Stable isotopes have also been used in hydrograph separation (Sklash 1990). Rivers have two principal sources of water: surface runoff, and groundwater. In rivers where runoff is the major source of water, large seasonal variations in $\delta^{18}O$ and δD are measured. In rivers where groundwater (baseflow) is the major source, these variations in $\delta^{18}O$ and δD are much less significant due to a much longer retention period and aquifer mixing.

c. Unstable isotopes.

(1) General. Unstable isotopes are of particular use in determining the age of water. Radioactive decay is the conversion of atomic mass to energy in the form of gamma rays, alpha particles, etc., over time:

$$-\frac{dM}{dt} = kM \tag{4-5}$$

where

M = mass of unstable isotope

k = decay constant

The half-life ($t_{1/2}$) of an isotope is defined as the time period in which half of the initial amount of unstable isotopes have decayed and can be calculated by the following formula:

$$t_{1/2} = \frac{\ln 2}{k} \tag{4-6}$$

The two most commonly used unstable isotopes in groundwater hydrology are ^{3}H (tritium) and ^{14}C (radiocarbon). ^{36}Cl (radiochloride) is also used for the dating of very old groundwater.

(2) ^{3}H (tritium). Tritium has a half-life of 12.3 years. Small amounts are produced naturally in the atmosphere. Between 1952 and 1969 nuclear testing in the atmosphere raised the tritium content in rainfall from 5-10 TU (tritium units) in the 1940's to 100-1,000 (or more) TU in the 1960's. The peak of tritium levels occurred in 1963 (Figure 4-10). Currently (in 1996) the tritium content in rainwater is approximately 10-30 TU. Tritium in groundwater is not significantly affected by chemical processes. Common uses of tritium analysis include:

(a) Distinguishing between water that entered into the aquifer prior to 1952 (pre-bomb), and water that was in contact with the atmosphere after 1953.

(b) Estimating recharge rates by locating the depth of the tritium peak which occurred in 1963 (Robertson and Cherry 1989).

(c) Estimating groundwater flow velocity.

Because of hydrodynamic dispersion, mixing of aquifer waters, and the variable tritium source, age estimates are best viewed as one more input in the formulation of a hydrogeologic conceptual model,

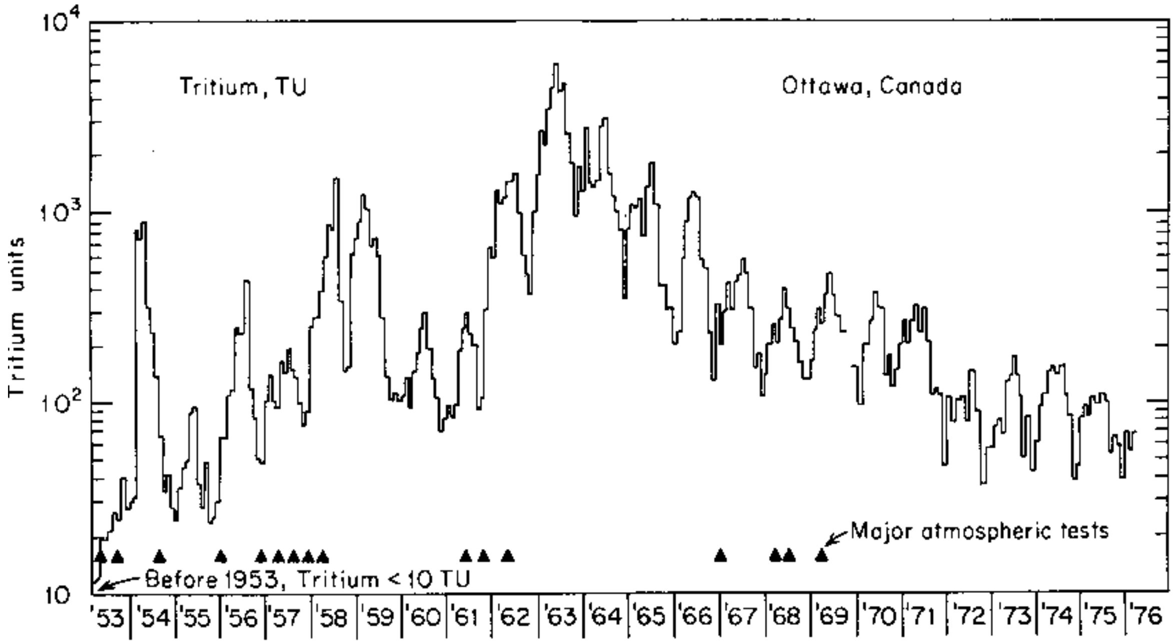

Figure 4-10. Variations of tritium (TU) in precipitation at Ottawa, Canada ("Groundwater," by Freeze/Cherry, © 1979. Reprinted by permission of Prentice-Hall, Inc., Upper Saddle River, NJ.)

rather than an exact determination of residence time (Smith and Wheatcraft 1992). Table 4-3 provides an estimate of groundwater residence time as of 1987 (Davis and Murphy 1987).

Table 4-3
1987 Relationship Between Tritium Concentration and Groundwater Age (Davis and Murphy 1987)

Concentration, TU	Interpretation (1987)
< 0.2	Water is older than 50 yr
0.2 - 2.0	Water is older than 30 yr
2 - 10	Water is likely at least 20 yr old
10 - 100	Water is less than 35 yr old, may be modern
> 100	Probably related to peak fall-out period of 1960-1965

(3) ^{14}C (radiocarbon). ^{14}C has a half-life of 5,730 years. Like tritium, small amounts of ^{14}C are also produced naturally in the atmosphere; and like tritium, much higher concentrations of ^{14}C were introduced into rainfall by atmospheric nuclear testing. Because of its longer half-life, ^{14}C can be a useful tool for dating groundwater as old as 30,000 years (Davis and Murphy 1987). Hydrogeologic settings with residence times of this magnitude include large-scale regional flow systems, and systems in thick low-permeability sediments. The measurement of ^{14}C along several points in a regional flow system allows for an interpretation of age differences, areas of recharge, and flow velocities. Complications which can occur when using radiocarbon analysis include:

(a) Dissolution of carbonate minerals or oxidation of organic matter may add "dead" (no detectable ^{14}C) carbon to water, giving an erroneously old age. A number of correction techniques exist. Phillips et al. (1989) review six methods and apply age dating as a tool on modeling groundwater flow in the San Juan Basin.

(b) Mixing of aquifer waters. A low ^{14}C content can be indicative of either old water, or a mixture of young water and "dead" water. Therefore, low ^{14}C measurements can be interpreted for age of groundwater only in flow scenarios where mixing is insignificant.

(4) ^{36}Cl (chloride-36). ^{36}Cl has a half-life of 300,000 years. ^{36}Cl is produced by cosmic ray interaction with the atmosphere. ^{36}Cl has been used to date very old groundwater up to 1 million years old (Phillips et al. 1986). Disadvantages are that abundance is very small, and the analytical techniques required are complex and expensive.

d. Approximate cost (1988) of isotopic analysis. Hendry (1988) made the following cost estimates for isotopic analysis:

Isotope	Cost
^{2}H	$40
^{18}O	$30
^{3}H	$45-$100[1]
^{14}C	$70-$215[1]

[1] Range of costs reflects degree of accuracy required.

Collecting samples of ^{18}O, ^{2}H, and ^{3}H is simple. ^{14}C is not as simple, but is relatively easy, requiring only the precipitation of the dissolved inorganic carbon.

e. Example of the use of isotopes in groundwater studies.

(1) Setting. The year is 1996. A disposal site is located above the water table in a shallow phreatic aquifer (Figure 4-11). This aquifer is underlain by a clay layer which appears to confine an underlying aquifer. The lower aquifer is used for domestic water supplies. Water-level measurements show that water is moving laterally from site A to site B in both aquifers. Hydraulic gradients also indicate that there is the potential for downward flow from the phreatic aquifer to the underlying aquifer.

(2) Question. Will the clay layer, which appears to separate the two aquifers between sites A and B, prevent flow from the upper aquifer to the lower one?

(3) Solution. The "non-isotopically inclined hydrogeologist" might answer: "How many test holes need to be drilled to make sure the clay layer is continuous?" and "What tests can we perform to determine the vertical hydraulic conductivity of the confining layer?" This can be at a considerable cost. However, the "isotopically aware hydrogeologist" would consider the following approach:

Use isotopic analysis to determine if such a connection exists.

(4) Hypothetical results:

(a) Tritium (^{3}H) concentrations more than 5.0 TU were encountered in the piezometers above and below the clay layer.

(b) Analysis: Post-bomb (1953) water has entered both the phreatic and underlying aquifer and the clay layer is not an effective barrier to water movement. Thus, any contaminants that might migrate from the waste site could enter the lower aquifer and contaminate local water supplies.

(c) Tritium (^{3}H) concentrations in piezometers in the deep aquifer were measured at less than 0.1 TU.

(d) Analysis: The age of the water is greater than 50 years, although this provides little information as to the degree to which the clay layer acts as a barrier.

(e) ^{14}C analyses were conducted in the deep piezometers at sites A and B, and results indicated age dates on the order of tens of thousands of years.

(f) Analysis: The clay layer is continuous and acts as a barrier to separate the phreatic aquifer from the deeper aquifer.

4-12. Response of Groundwater Levels to Loading Events

a. General. It is possible to estimate values of hydraulic properties of aquifers from the frequency response of the water-level fluctuations. Short-term fluctuations in confined aquifers can be caused by changes in atmospheric pressure of the atmosphere, earth tides, seismic events, and external loads such as passing trains. In many cases, there may be more than one mechanism operating simultaneously, with atmospheric pressure changes and earth tidal fluctuations being the two common natural events. These fluctuations offer evidence that confined aquifers are not rigid bodies, but are elastically

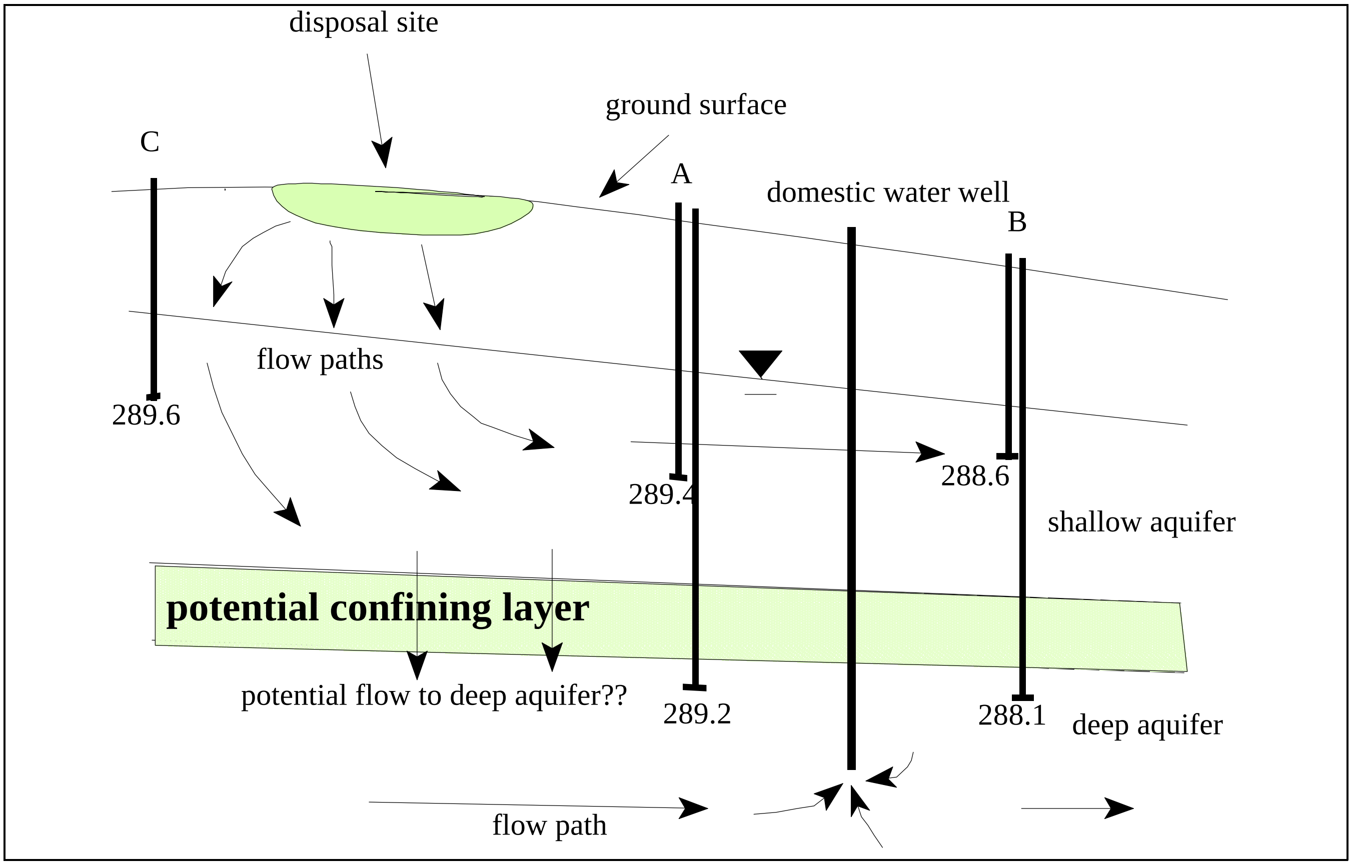

Figure 4-11. Hypothetical proposed disposal site

compressible. Earth tides can lead to water-level changes of 1 or 2 cm; atmospheric pressure changes may cause fluctuations of several tens of centimeters, depending upon the elastic properties of the aquifer and the magnitude of change in atmospheric pressure. Hydraulic properties, such as storage coefficient (storativity) and porosity, can be calculated from the response of measured water levels to these natural loading events. These types of water-level changes are damped in unconfined aquifers.

b. Effects of changes in atmospheric pressure on water levels.

(1) General. Although variations in barometric pressure have no significant effect on the water table levels in unconfined aquifers, the variations do cause well water levels for confined aquifers to fluctuate greatly. This is explained by recognizing that aquifers are elastic bodies. In a confined aquifer, an increase in atmospheric pressure is transmitted directly to the water surface of the piezometer, tending to displace water from the piezometer into the aquifer. On the other hand, the increased atmospheric pressure also increases the load on the confined aquifer, which tends to displace water from the aquifer into the piezometer. Part of this increased atmospheric load is born by the mineral skeleton of the aquifer, however, and the net result of an increase in barometric pressure is to decrease the water level in the piezometer. Conversely, decreases in atmospheric pressure produce increases in piezometer water levels. For an unconfined aquifer, atmospheric pressure changes are transmitted directly to the water table, both in the aquifer and a well; hence, no fluctuation results. Thus, when measuring water levels in confined aquifers, the effect of atmospheric pressure should be considered.

(2) Barometric efficiency. In confined aquifers, when atmospheric pressure changes are expressed in terms of columns of water, the ratio of water level change to pressure change expresses the barometric

efficiency of an aquifer. In Figure 4-12, the upper curve indicates observed water levels. The lower curve shows atmospheric pressure (inverted) in feet of water and multiplied by 0.75. A close correspondence of major fluctuations exists in the two curves. Thus, Figure 4-12 illustrates a measured barometric efficiency of approximately 0.75 for an aquifer. Typical barometric efficiencies in confined aquifers range from 0.20 to 0.75. The units of measurement (SI or metric) for water levels and atmospheric pressure used in Figure 4-12 are not important, but they must be consistent with each other.

(a) To convert the values on the right y-axis of Figure 4-12 into values we are familiar with from the evening news weather report, the units must be changed from feet of water to inches of mercury, considering the factor of barometric efficiency:

(ft of water) (12in./ft) (1 in. Hg/13.6 in. water) (1/barometric efficiency) = in. Hg

In metric units, (Hg, inches)(2.54) = (Hg, centimeteters)

(b) Barometric efficiency can be defined mathematically by:

$$B = \frac{\Delta h \gamma_w}{\Delta p_a} \tag{4-7}$$

where

B = barometric efficiency of an aquifer

Δh = change in measured water level

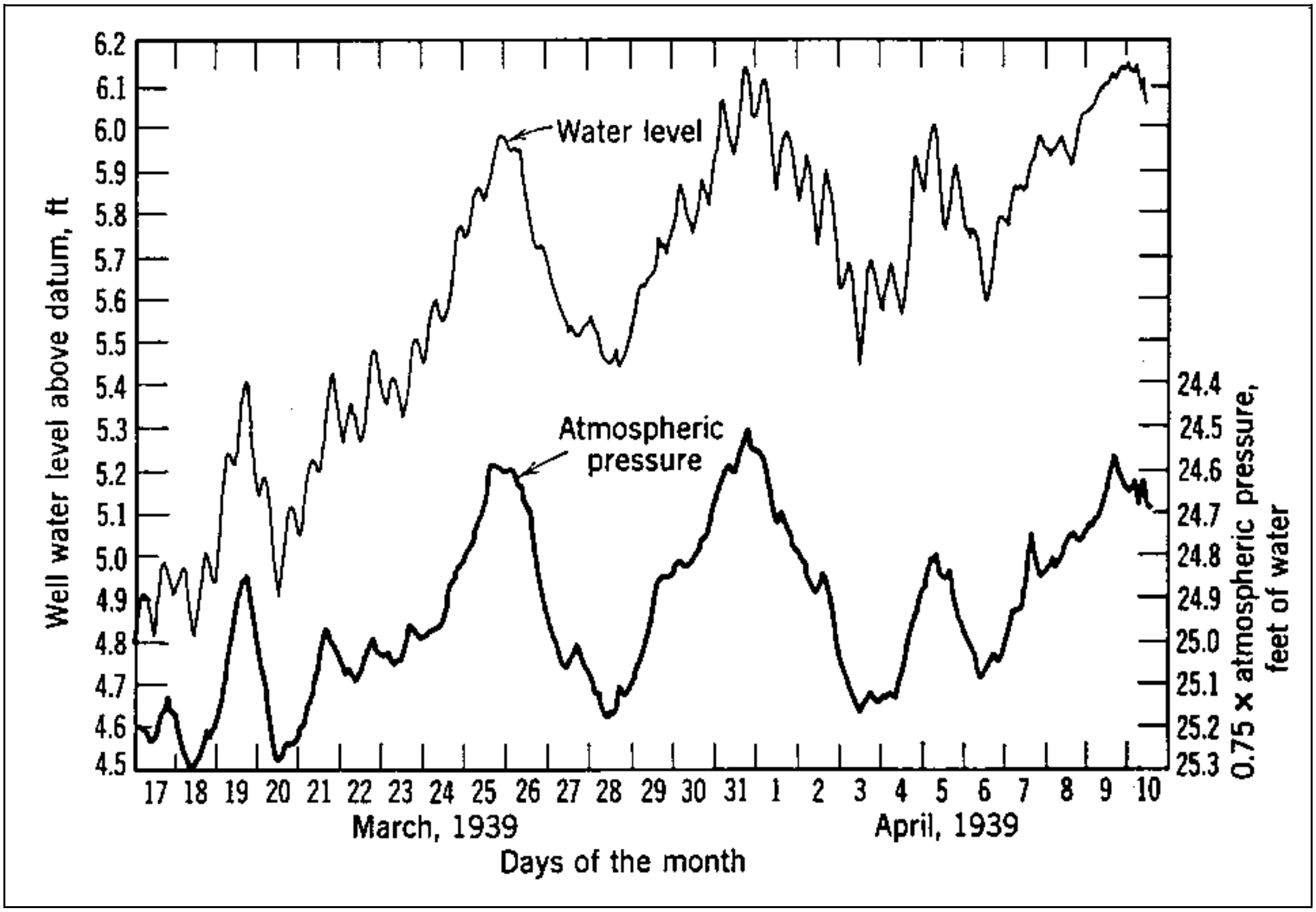

Figure 4-12. Response of water level in a well penetrating a confined aquifer to atmospheric pressure changes. (Robinson, ©1939, reprinted by permission of the American Geophysical Union)

γ_w = specific weight of water

Δp_a = change in atmospheric pressure

(c) In confined aquifers, well measurements can be corrected to a constant atmospheric pressure by:

- Deriving barometric efficiency.
- Correcting changes in water levels for corresponding changes in atmospheric pressure.

(d) Jacob (1940) derived an expression for relating the barometric efficiency of an aquifer to aquifer properties:

$$B = \frac{nE_s}{nE_s + E_w} \tag{4-8}$$

where

B = barometric efficiency of an aquifer

n = porosity

E_w = bulk modulus of compressibility for water

E_s = modulus of elasticity of the aquifer solids

(e) Thus, barometric efficiency is directly proportional to the rigidity of an aquifer. Barometric efficiency approaches one for rigid aquifers, and is small for flexible unconsolidated aquifers. A barometric efficiency of one suggests that the well is a perfect barometer, in which all changes in stress on the aquifer are borne by the mineral skeleton. The right side of Equation 4-8, and therefore barometric efficiency, is constant for a given aquifer.

(3) Relationship between barometric efficiency and storage coefficient. The compressibility of an aquifer can be expressed as:

$$\beta = \frac{n}{E_w} + \frac{1}{E_s} \tag{4-9}$$

where

β = aquifer compressibility

n = porosity

E_w = bulk modulus of compressibility for water

E_s = modulus of elasticity of the aquifer solids

(a) The storage coefficient of a confined aquifer can be defined as:

$$S = \beta\gamma_w b \tag{4-10}$$

where

S = storage coefficient

β = aquifer compressibility

γ_w = specific weight of water

b = aquifer thickness

(b) By combining Equations 4-8, 4-9, and 4-10, a relationship between barometric efficiency and aquifer storage coefficient can be derived:

$$S = \frac{n\gamma_w b}{E_w B} \tag{4-11}$$

where

S = storage coefficient of the confined aquifer

n = porosity

γ_w = specific weight of water [1,000 kg/m^3]

b = aquifer thickness [m]

E_w = bulk modulus of compression of water [2.07×10^9 N/m^2]

B = barometric efficiency of the aquifer

(4) Example problems.

(a) If the confined aquifer in Figure 4-12 is 40 m thick and has a porosity of 0.2, what is its estimated storage coefficient?

$$S = \frac{n\gamma_w b}{E_w B}$$

S = (0.2)(1,000)(30)(9.8)/(2.07 × 10^9)(0.75) = 3.8 × 10^{-5}

Note: the value of 9.8 (gravity) in the above equation is required to convert from units of Newtons to units of kilograms.

(b) As illustrated in Figure 4-13, well A and well B are screened beneath a confining layer. Well A was measured on 3/1/90 to be 301.0 ft above mean sea level (msl); well B was measured on 3/3/90 to be 300.75 m above msl. The barometric efficiency of the aquifer is 0.50. When well A was measured, the barometric pressure was 756 cm (of water column). When well B was measured, the barometric pressure was 806 cm (of water column). What is the gradient between the two wells?

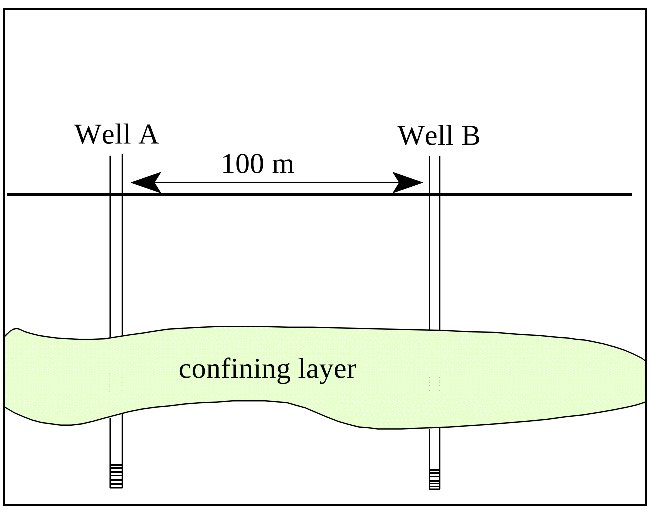

Figure 4-13. Hypothetical well locations

The change in barometric pressure = 50 cm of water column.

The barometric efficiency = 0.5.

Therefore, the change in water levels = 0.25 m; therefore, the gradient = 0.

c. Earth tides.

(1) General. Water level changes in response to tidal fluctuations can occur in confined aquifers as a response to gravity, or in confined aquifers which outcrop to the ocean. In the latter case, changes in pressure heads due to tides are transmitted directly to the water in the aquifer at the outcrop. In confined aquifers not adjacent to the ocean, the effect of gravitational forces on water levels is a function of the rigidity of the aquifer. As discussed in the previous section, atmospheric pressure acts not only on the rock matrix and its contained water, but also on the water level in an open observation well. Gravitational forces act only on the rock matrix and its contained water. Earth tides are predominantly the result of the gravitational pull of the moon; and to a lesser extent, the result of the gravitational pull of the sun. Earth tides cause small (1-2 cm) water-level fluctuations in wells located in confined aquifers.

(2) Tidal efficiency. The tidal efficiency of an aquifer is defined as the ratio between change in head in a confined aquifer and change in tidal force:

$$T_e = \frac{\Delta h \gamma_w}{\Delta F_t} \tag{4-12}$$

where

T_e = tidal efficiency of an aquifer

Δh = change in measured water level [ft]

γ_w = specific weight of water

ΔF_t = change in tidal force

The tidal efficiency is inversely proportional to the rigidity of an aquifer. Tidal efficiency approaches zero for rigid aquifers, and approaches one for flexible unconsolidated aquifers. A tidal efficiency of one suggests that the well is perfectly flexible, in which all changes in stress on the aquifer are born by the pore water. The relationship between barometric efficiency *B* and tidal efficiency T_e is:

$$B + T_e = 1 \tag{4-13}$$

4-13. Conclusion

Investigation of the subsurface is a dynamic and inexact science; but is essential to the success of a groundwater study. Aquifer characterization is dependent upon the quality and quantity of data gathered and the interpretation of that data to obtain a good understanding of the hydrogeologic setting. Often, site characterization studies focus on pumping tests and borehole geophysics, and neglect other valuable and cost-effective investigative methods such as cone penetrometers, surface geophysics, and isotopic analyses. Due to time and financial constraints, it is important for the study manager to be familiar with all potential sources of data, and plan the site investigation in the most efficient manner to meet study objectives.

Chapter 5
Computer Modeling of Groundwater Flow

5-1. General

a. Chapter organization. In Chapter 3, an overview of planning and management considerations in performing a groundwater site characterization and modeling study was presented. This chapter provides a technical overview of the theory, development, and use of computer models for simulating groundwater flow. Initially, the criteria to be used in the selection of a computer code are discussed. An overview of the components of a groundwater model is then presented, followed by a discussion on model calibration, execution, and interpretation of results.

b. What is a groundwater model? A groundwater model is a replica of some real-world groundwater system.[1,2,3,4] A groundwater model can be as simple as a construction of saturated sand packed in a glass container or as complex as a three-dimensional mathematical representation requiring solution of hundreds of thousands of equations by a large computer. The term "modeling" refers to the formation of conceptual models and manipulation of modeling software (codes) to represent a site-specific groundwater system. The resulting representation is referred to as a "model" or a "model application." The accuracy of a model is dependent upon the level of understanding of the system the model is to represent. Thus, a complete site investigation and accurate conceptualization of site hydrogeology are necessary precursors to a successful modeling study.

c. Components. As discussed in Chapter 3, basic components of a groundwater modeling project are:

(1) A statement of objectives.

(2) Data describing the physical system.

(3) A simplified conceptual representation of the system.

(4) Data processing and modeling software.

(5) A report containing written and graphical presentations.

d. Protocol. General protocol for performing modeling studies is discussed in Chapter 3, and typically follows a process that includes the following steps:

(1) Determination of modeling objectives.

(2) Data gathering and organization.

(3) Development of a conceptual model.

(4) Numerical code selection.

(5) Assignment of properties and boundary conditions to a grid.

(6) Calibration and sensitivity analysis.

(7) Model execution and interpretation of results.

(8) Reporting.

The following sections in this chapter will focus on steps 4-7; i.e., the technical aspects of developing a computer model of groundwater flow.

5-2. Code Selection

a. Identifying needs. Selecting the appropriate code for a modeling job involves matching modeling

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[1] The International Groundwater Modeling Center defines a model as "a non-unique, simplified, mathematical description of an existing groundwater system, coded in a programming language, together with a quantification of the groundwater system, the code simulates in the form of boundary conditions, system parameters, and system stresses" (U.S. Environmental Protection Agency (USEPA) 1993).

[2] "A model is a simplified description of a physical system" (U.S. Department of Energy 1991).

[3] "A groundwater flow model is an application of a mathematical model to represent a site-specific flow system" (ASTM 1992).

[4] "A mathematical model is a replica of some real world object or system" (Nuclear Regulatory Commission 1992).

needs with the capabilities and controls of available codes. Before selecting a code for use, the modeling objectives, the conceptual model, and project time and cost constraints should be well-defined. Use this information to develop a list of needs. Purchasing a code first, then defining the problem second may cause insurmountable problems. Table 5-1 lists some questions helpful in determining needs and matching these with appropriate codes.

Table 5-1
Determination of Model Needs

Code attributes
What is the general type of problem to be solved (flow in an unconfined aquifer, flow in a fractured confined aquifer, well field design)?
Does the code have the capability to adequately model the hydrologic/geologic features of the site (i.e., wells, rivers, reservoirs, precipitation, watershed runoff, evapotranspiration, variable-density flow, vertical gradients, faults, etc.)?
What are the dimensional capabilities needed (1-D, 2-D Horizontal, 2-D Vertical, quasi 3-D, 3-D)?
What is the best-suited solution method (analytical, finite difference, integrated finite difference, finite element, matrix solver)?
Is a particular mathematical basis needed (empirical vs. mechanistic, deterministic vs. stochastic)?
What grid discretization features are needed?
Will unusual grid size or computational capabilities be required?
What pre- and post-processors are available?
Code administration
Who developed, distributes, and supports the code?
What is the quality of the support?
What is the quality of the user's manual?
What is the cost?
Is the code proprietary?
Is a list of user references available?
Is the code widely used and well verified?

b. Types of codes. Four ways of describing groundwater models are (USEPA 1993):

(1) Objective-based: groundwater supply, well field design, prediction, parameter estimation, and education models.

(2) Process-based: saturated flow, unsaturated flow, contaminant transport, and flow path models.

(3) Physical-system-characteristics based: unconfined aquifer, confined aquifer, porous media, fractured rock, steady-state, time varying, multi-layer, and regional scale models)

(4) Mathematical-based: dimensionality of solution equations, analytical, numerical, empirical, deterministic, and stochastic models.

The above categories are not exclusive. Typically, a model application is labeled using a combination of adjectives from the above categories; for example, a "two-dimensional transient numerical model of groundwater flow in porous media for the prediction of flow paths" is one possible label.

c. Solution methods. Differing solution methods affect the difficulty of use and overall flexibility of modeling software. The three most common solution methods used in groundwater modeling, listed in increasing complexity are: analytical, finite difference, and finite element. Each method solves the governing equation of groundwater flow and storage, but differ in their approaches, assumptions, and applicability to real-world problems.

(1) Analytical methods. Analytical methods use classical mathematical approaches to resolve differential equations into exact solutions. They provide quick results to simple problems. Analytical solutions require assumptions of homogeneity and are limited to one-dimensional and two-dimensional problems. They can provide rough approximations for most problems with little effort. For example, the Thiem equation can be employed to estimate long-term drawdown resulting from pumping in a confined aquifer.

(2) Finite difference methods. Finite difference methods solve the partial-differential equations describing the system by using algebraic equations to approximate the solution at discrete points in a rectangular grid. The grid can be one-, two-, or three-dimensional. The points in the grid, called nodes, represent the average for the surrounding rectangular block (cell). Although adjacent nodes have an effect on the solution process, the value for a particular node is distinct from its neighboring nodes. Grids used in finite difference codes generally require far less set-up time than those of finite element codes, but have less flexibility in individual node placement. Many common codes, such as MODFLOW (McDonald and Harbaugh 1988), use the finite difference solution method.

(3) Finite element methods. Finite element methods differ from finite difference methods in that the area (or volume) between adjacent nodes forms an element over which exact solution values are defined everywhere by means of basis functions. A main practical difference is that finite element codes allow for flexible placement of nodes which can be important in defining irregular boundaries. However, defining a unique location for each finite element node requires a more labor-intensive grid setup than that of finite difference. FEMWATER (Lin et al. 1996) is a common code using the finite element solution method.

d. *Code references.* Selection of a code ideally requires knowing the capabilities, attributes, and nuances of all available codes, then selecting the most suitable one. There are numerous commercial codes for use in groundwater modeling. Practically, the modeler often lacks up-to-date information on all available codes and lacks sufficient time to sort through code details. An extensive list of codes, their respective characteristics and contact addresses, and an assessment of their usability and reliability is found in the “model information database” of the International Groundwater Modeling Center. A selected listing from that database is found in “Compilation of Groundwater Models” (USEPA 1993). Additional help in selecting a short list of potential codes can be provided by various publications and databases provided by professional organizations and institutes such as the National Groundwater Association, the International Groundwater Modeling Center, and research offices of the CE, USEPA and USGS, among others.

e. *Pre- and post-processors.*

(1) General. Some pre-processors allow superposition of the grid and the site map, and then allow interactive assignment of boundary conditions, aquifer properties, etc. Post-processors allow the numerical output to be presented as contour maps, raster plots, flow path plots, or line graphs. Choosing a code that does not have, or cannot be easily linked to, pre-and post-processors should be avoided. Hundreds of simulation runs are typically performed for a modeling job, each requiring adjusting input files and interpreting results. Lack of tools to aid in these tasks can cumulatively result in large amounts of additional time spent. An effective link to quality output graphics is critical because many modeling results are best presented pictorially. Systems that include groundwater modeling as just one application in an overall data modeling and representation system are being developed. Such systems reduce overall modeling time by reducing manual data manipulation requirements.

(2) The Department of Defense Groundwater Modeling System. The Department of Defense Groundwater Modeling System (GMS) provides a comprehensive graphical environment for numerical modeling, tools for site characterization, model conceptualization, mesh and grid generation, geostatistics, and sophisticated tools for graphical visualization. Several types of numerical codes are supported by GMS. The current (1996) version of GMS provides a complete interface for the codes MODFLOW, MT3D (a contaminant transport model), and FEMWATER (a finite element model). Many other models will be supported in the future. Tools and features of GMS include the following:

(a) Graphical user interfaces to MODFLOW, MT3D, and FEMWATER groundwater flow and transport codes.

(b) Site characterization using solid modeling of earth masses defined from borehole data.

(c) Surface and terrain modeling using Triangular Irregular Networks.

(d) Automated two- and three-dimensional finite element and finite difference grid generation.

(e) Geostatistical tools for two-dimensional interpolation and three-dimensional interpolation of scattered data, including kriging and natural neighbor interpolation.

(f) Three-dimensional graphics, including contours, vector arrows, shaded images, iso-surfaces, cross sections, and cut-away views.

(g) Animation of steady-state and transient data.

(h) Site maps can be displayed simultaneously with model simulation results.

(i) Intuitive and modular user interface takes advantage of graphical display, and point and click editing.

(j) Available for MS Windows and UNIX platforms.

5-3. Initial Model Development

a. Basic components. After construction of the conceptual model (Chapter 3) and selection of the modeling software, the features of the conceptual model are transferred to an input file that defines the mathematical model. Boundary conditions, grid dimensions and spacing, initial aquifer properties, and time-stepping features are specified according to the particular requirements of the selected code. Input file development can be expedited by use of a pre-processor that allows direct assignment of values to a grid that is superimposed on a site map. At the end of this initial development phase, the model will be ready for calibration.

b. Boundary conditions.

(1) Boundary conditions are constraints imposed on the model grid that express the nature of the physical boundaries of the aquifer being modeled. Boundary conditions have great influence on the computation of flow velocities and heads within the model area. Three types of boundary conditions are commonly used in groundwater flow models:

(a) Specified head. A specified head boundary can be used when expressing the constraints imposed by a lake, a reservoir, or a known phreatic surface. Head data can be measured much easier than flux data, making specified head boundary conditions more desirable for natural features that vary over the length of boundary or vary through time. One caution is that a specified head boundary allows an inexhaustible amount of water flow.

(b) Specified flux. A specified flux boundary expresses the effects of a feature that constrains flow into or out of a boundary or a location where the flux can be estimated. Examples include: zero flux from a subsurface barrier, surface infiltration, leakage across a confining layer, or a "no-flow" boundary chosen to coincide with a groundwater divide or a groundwater flow line so that lateral flux is negligible. Caution should be used in the latter case because natural groundwater divides and "no-flow" lines can move when the aquifer is stressed.

(c) Value-dependent flux. A value-dependent flux allows flux through the boundary according to some external constraint. Examples include infiltration from a pond dependent upon pond levels, and injection of well water dependent upon injection pressure. This type of boundary is used commonly in transient simulations.

(2) Boundary location and orientation. The type of boundary chosen should be fully consistent with the water budget and boundary conditions identified in the conceptual model. Choosing an observable natural feature such as a lake, river, or a groundwater divide as a grid boundary allows the boundary condition to approximate a constraint that can be quantified by measurement (reservoir levels) or reasonable estimate (flux across a groundwater divide). When a natural feature is not available, orienting the boundary to run parallel with a groundwater flow line allows for designation of a boundary with a specified flux of zero. Although the boundaries can be placed anywhere, wise placement reduces uncertainty, thus contributing to more realistic model outcome.

(3) Boundary type variation. Simple models often have uniform conditions for each whole boundary. More detailed models often have boundaries broken into subregions having varying values or differing types of boundary conditions altogether. The type of boundary conditions applied can greatly affect modeling results. A study on boundary condition effects showed that three

groundwater models, the same in all respects except for their boundary conditions, responded very differently to an imposed stress. The study emphasized that when calculated heads match those of the natural system, it does not guarantee that the model boundary conditions match those of the natural system (USGS 1987).

(4) Boundary and system stresses. The location and magnitude of stresses applied to the model affect the appropriate choice of boundary conditions. For example, if a groundwater divide is chosen as a zero-flux boundary condition, the natural boundary and the model boundary may match closely in an unstressed steady state. If, however, an extraction well is placed near this boundary in the computer simulation, the original flow system is no longer being modeled and the original boundary condition and its alignment may need to be changed. A rule of thumb is to avoid placing boundaries close to where stresses will be applied.

(5) Water table boundary. The water table boundary is typically specified three ways: (a) as a dependent variable using the Dupuit assumptions (commonly used in two-dimensional and three-dimensional applications), (b) as a designated no-flow boundary (usually used in three-dimensional and profile applications), or (c) as a dependent variable in an unsaturated/saturated model application. The Dupuit assumptions are: that flow in an unconfined aquifer is horizontal, the head does not change with depth, and that horizontal flow is driven by the water table gradient at all depths. Codes using the Dupuit assumptions allow for treating the water table as the feature to be computed by the model which is often exactly what is desired. The response of the water table to pumping from a well or variations in reservoir stages can be solved with codes using the Dupuit assumptions. Generally speaking, codes using the Dupuit assumptions are more simple and less labor intensive than those requiring the water table to be designated (fixed). Codes requiring an unsaturated/saturated zone interface have complex and detailed requirements and are generally only used for localized applications because of the detailed definition required.

c. System recharge and withdrawal stresses. Groundwater models are useful in predicting the effects from special recharge and withdrawal stresses, usually injection and extraction wells, that cause a relatively large water exchange in a relatively small area. These analyses can predict general aquifer response to special stresses. However, another method, such as spreadsheet analysis of well drawdown equations, is necessary to simulate the local effect of pumping mainly because node spacing in most site models is typically many times greater than the diameter of the well.

d. Grid design. Model grids discretize the continuous natural system into segments (i.e., cells, elements, blocks) that allow numerical solutions to be calculated. The grid should be superimposed on a map of the area to be modeled. Grid boundaries should be located consistent to the conceptual model and following the guidelines discussed in the boundary condition section. In finite difference modeling, grid nodes lying outside the boundary are often designated as non-computational to minimize computation volume. When designating boundary nodes, the modeler must be aware of whether the modeling software uses a block-centered or mesh-centered convention and place the nodes accordingly. The flux boundary for the mesh-centered nodes is calculated on the line (or plane) directly between the nodes. The flux face is calculated at the midpoint between the nodes when using the block-centered convention. Flexible placement of finite element boundary nodes allows exact placement of nodes along the boundary.

e. Grid resolution and geometry. The following guidelines should be followed when constructing a numerical model grid:

(1) Node spacing. The spacing between nodes, called grid resolution, should be responsive to sharp changes in physical features, temporal conditions, and, numerical stability and overall model size constraints. Generally, node spacing is finer where the dependent variables, usually the hydraulic gradient and flux, are subject to greater change. The areas near extraction wells, infiltration trenches, and confined aquifer flow channels are examples. Finer node placement may also be required where curved surfaces or irregular boundaries are being represented. Where definition of irregular surfaces is required, use of a code not allowing for flexible node placement should be questioned as it could result in a grid with an excessively large number of nodes. Sensitivity to grid resolution should be checked when performing a thorough analysis because differing grid resolutions can affect modeling results.

(2) Selection of model layers. In three-dimensional models, model layers allow for the simulation of flow in separate hydrographic units, leakage between aquifers, and vertical flow gradients. Typically, one model layer is selected for each hydrostratigraphic unit; however, if there are significant vertical head gradients, two or more layers should be used to represent a single hydrostratigraphic unit (Anderson and Woessner 1992).

(3) Avoiding numerical errors. Numerical error and unintended biases in solution of the flow equations can be minimized by avoiding large variations in node spacing and large aspect ratios. The aspect ratio is the maximum dimension of a block or element divided by the minimum dimension. An aspect ratio of one is usually ideal for minimizing numerical errors. As a rule of thumb, aspect ratios up to 10:1 in non-sensitive areas of a grid are usually acceptable and expanding block or element sizes by 1.5 times the adjacent block sizes should be avoided.

(4) Grid sizes. The overall size of the grid (i.e., total number of nodes) should be adequate to define the problem and produce results consistent with modeling objectives, but not so large as to cause excessive run preparation and computation requirements. Several hundred iterations of adjusting the model input, running the model executable code, and interpreting the results are often required in a modeling job. An excessively large grid will expand the time requirements for each iteration, resulting in a cumulatively large impact to the modeling quality or schedule.

f. Initial conditions. Initial conditions refer to the values of the dependent variables defined at the beginning of the simulation. For steady-state models (no time variation), initial conditions need only approximately match the natural system because the solution for each dependent node can be found eventually through repeated iteration. In contrast, transient models (time variation included) require initial conditions closely matching natural conditions at the beginning of the simulation. To do this, it is often necessary to first run a steady-state model, or alternately, run the transient model for a lead-up period of time before beginning the interval of interest. Transient models commonly have boundary conditions that vary as the model simulates an aquifer system response through time. A seasonally fluctuating lake level is an example, and could be simulated using specified head nodes that vary according to some predetermined schedule.

g. Aquifer material properties. Aquifer material properties refer to those aquifer properties, such as hydraulic conductivity and anisotropy, that govern flow rate and flow direction. Table 5-2 presents basic aquifer properties and typical data sources.

Table 5-2
Aquifer Properties and Data Sources

Hydraulic conductivity (pumping tests, slug tests, slug interference tests, grain size analysis, laboratory permeameter tests, tracer tests).
Transmissivity (pumping tests, calculation from hydraulic conductivity).
Porosity (grain size analysis, observation at trenching or outcrop sites, geophysical tests).
Anisotropy (tracer tests, geologic conceptualization and history).
Aquifer storage (pumping tests, geophysical methods).

h. Assignment of aquifer material properties to grid. The aquifer properties previously listed are assigned throughout the model grid by use of a preprocessor or directly into an input file. A simple model may assign uniform hydraulic conductivity in all nodes while complex models may have many different node groups, layers, or zones, each with differing conductivity values. The discretization of zones of homogeneous aquifer properties should be based primarily on site geology. The discretization of zones based on water levels should only be considered in areas where a high quantity (and quality) of data presents compelling physical evidence of distinct hydrogeologic conditions. Geostatistical methods may be employed to distribute the properties to all nodes based on the data known at only a few nodes. However, geostatistics provides a systematic method for distributing the properties and does not account for site geological conditions. The total number of zones of homogeneity should be kept at the minimum required to adequately represent the system within data constraints.

i. Representing uncertainty. The inherent uncertainty in the information describing aquifer properties should be recognized and preserved throughout the analysis. Most properties should be represented as ranges because of the uncertainty associated with gathering, interpreting, and extrapolating the data to the model. Aquifer properties are usually gross, large-scale representations of properties that are increasingly variable when viewed at increasingly smaller scales. Dealing with uncertainty in model inputs is discussed in Section 5-6 on modeling application.

j. Time-stepping. Time-stepping is the discretizing of the flow equations through time and is used in transient simulations. Like node spacing, time-stepping should be fine enough to define the problem adequately, but not too small to exceed practical computation constraints. Time-stepping should be finer at those times when new stresses are introduced. Changes in boundary conditions usually control the time-step requirement. Initial time-stepping designation should be estimated by experience and refined with a time-stepping sensitivity analysis. Some codes combine time-steps into groups called stress periods.

5-4. Model Calibration and Sensitivity Analysis

a. Calibration defined. Calibration is the process of adjusting model inputs to achieve a desired degree of correspondence between the model simulations and the natural groundwater flow system. A flow model is considered calibrated when it can reproduce, to an acceptable degree, the hydraulic heads and groundwater fluxes of the natural system being modeled. This is accomplished by finding a set of values for the boundary conditions, aquifer properties, and stresses that result in computed heads and fluxes matching their natural counterparts at target locations. In other words, calibration methods solve a problem inversely by iteratively adjusting the unknowns (hydraulic conductivities, certain boundary fluxes, etc.) until the solution matches the knowns (usually the hydraulic heads). Multiple calibrations of the same system are possible using different boundary conditions and aquifer properties. There is not one unique calibration that is “correct” for any model because exact solutions cannot be computed with this multi-variable approach. Furthermore, because model zones of homogeneous aquifer properties should have a strong physical basis, the most accurate model is often not the model which most closely simulates calibration targets. At the end of the calibration process, the model should be ready for use to simulate the flow system.

b. Calibration methods. Methods of calibrating can be grouped into two categories: manual trial-and-error calibration and automated calibration. The state of the practice is that most modeling is performed by trial and error while automated methods are becoming increasingly usable and accepted. The method is code-dependant. Advances in modeling software allowing for greater use of automated calibration are expected.

(1) Manual trial and error. This method of calibration is labor-intensive. The modeler makes successive cycles involving interpreting prior results to determine where inputs need adjustment, making speculative adjustments to the input code, re-running the model and output software, and then comparing the computed results to the natural system. Typically, hundreds of iterations are made before an acceptable calibration is achieved, the specific number depending on model complexity, experience of the modeler, and the acceptableness criteria applied. Typically, the inputs being adjusted are hydraulic conductivity (or transmissivity), storage, leakage across a confining layer used as a boundary, flux to and from a surface water body, and designation of boundary conditions. A typical manual trial-and-error calibration process includes the following steps:

(a) Complete initial model development and assignment of properties as outlined in this chapter.

(b) Identify the parameters to be adjusted during calibration and the appropriate range for each. These are determined from the initial sensitivity analysis and from the conceptual model.

(c) Identify the locations and values for the target points forming the calibration set. Groundwater flow models are usually calibrated to a set of observed potentiometric head levels.

(d) Iteratively run the modeling software and adjust input parameters until an acceptable match between observed and calculated values at the target points is achieved. If the model is being calibrated to a set of observed head values, the computed and estimated boundary fluxes must also be compared.

(e) Repeat steps (c) and (d) for different calibration conditions if desired. For example, a model can be calibrated to the seasonal low and seasonal high calibration conditions or to conditions where the aquifer is stressed by pumping or injection.

(2) Automated calibration. This method utilizes an objective function, such as minimization of the sum of the squared differences between observed and computed heads (residuals), to govern automatic iterative adjustment of values that would otherwise be adjusted manually. Automated codes do this in a systematic fashion and typically require constraints on sets of input values in the form of probability functions, conditional bounds, or weighted values. These constraints require the modeler to better define the uncertainty and variation within parameters, such as hydraulic conductivity, before code execution begins. Particular requirements for automated calibration codes vary.

(3) Comparison of calibration methods. Automated calibration methods have some potential advantages over trial-and-error methods. They can provide a systematic approach to calibration, allowing for efficiencies within individual modeling jobs and a basis for comparison between different modeling jobs. Statistical measurements are available from some automated approaches that are not usually performed in trial-and-error approaches. And finally, practitioners report that, because less time was spent on manual iteration, more time was available to refine the calibration and explore model sensitivity to various calibration options.

c. Matching computed values with target values. A key to calibration is the comparison of computed values, usually the computed heads, with observed values, often called "target values," to determine the appropriateness of the calibration. Questions to be considered when compiling a set of target values include the following:

(1) Do the target values reflect a steady-state or transient condition?

(2) Are there effects from local anomalies?

(3) Are the wells screened comparably?

(4) Are the measurement errors acceptable?

These questions can usually be answered by having a complete conceptual model and observing the changes to the set of target values over time.

d. Types of comparisons.

(1) Spatial graphic comparisons. This method often uses superimposed contoured water table surfaces or raster plots to show locations and magnitude of the differences between computed and observed values (residuals). These methods provide the modeler with an understanding of spatial variation of the residuals and can be key to selecting where further input parameter adjustments are required.

(2) Tabular comparison at target nodes. This method provides a quantifiable comparison of values point by point.

(3) Lumped-sum comparison. These methods lump residual measurements into single values and often take the form of: (a) mean error of the residuals, (b) absolute mean error to the residuals, and (c) root-mean-squared error of the residuals. Using the root-mean-squared error method provides a commonly used overall comparison.

e. Calibration cautions. Successful calibration to one model component does not guarantee a sound model.

(1) Head calibrations and boundary conditions. When model head results match observed head results, the groundwater flow system is not necessarily simulated accurately. As discussed in the "Boundary Conditions" section, research shows that models differing only by their boundary conditions can be calibrated to the same hydraulic head set, yet perform differently when stressed. Similarly, models that differ only by magnitude of hydraulic conductivity values can be calibrated to the same water table head set, yet produce differing flow velocities and boundary fluxes. These potential difficulties can be overcome to some degree by development of a sound conceptual model and ensuring the mathematical model appropriately represents key conceptual components. Estimated fluxes should be compared with calculated fluxes for any calibration using hydraulic heads as target values. Boundary conditions play an important role in soundness of modeling.

(2) Experience required. Model calibration requires extensive knowledge of the natural groundwater system being modeled. Understanding how to best achieve an adequate calibration and when the match between results is “good enough” depends on modeling objectives and expectations of the customers. Freyberg (1988) documents a study where nine groups, using the same model and input data, individually calibrated the model and produced widely varied final results. “The group achieving the best prediction chose to zone the conductivity field into a relatively few homogenous regions, while the group producing the worst prediction chose to ‘tweak’ the conductivity field grid block by grid block to achieve a good (in fact, the best) local fit to the observed data.” This study showed that an apparently “good” calibration does not necessarily result in accurate predictive results for other simulations.

f. Sensitivity analysis defined. A sensitivity analysis is a quantitative evaluation of the influence on model outputs from variation of model inputs. A sensitivity analysis identifies those parameters most influential in determining the accuracy and precision of model predictions (USEPA 1992). During sensitivity analysis, numerous model runs are performed, each having only one parameter varied by some specified percentage. Both positive and negative variance is tested.

g. Use of sensitivity analyses. Sensitivity analyses can be used to aid in model construction by identifying inputs requiring more definition. For example, the sensitivity analysis may show that existing hydraulic conductivity data ranges so widely that additional pumping tests are needed to obtain the desired level of accuracy in modeling results. Sensitivity analyses also aid in interpreting results. For example, uncertainty about the head values at a boundary may not be a concern if the analysis shows that output of interest is insensitive to these head values. Typically the analysis will show that sensitivity of groundwater flow to variation in hydraulic conductivity is relatively high.

h. Level of effort. Commonly, a small-scale analysis is performed during early model calibration, as a calibration aid; then a more rigorous analysis is performed after calibration as an indicator of model performance. If, for example, the results from a sensitivity analysis show that computed velocities vary ± 20 percent due to the reasonably expected range of site hydraulic conductivities, then interpretation of final model results should reflect this.

5-5. History Matching

Following calibration and sensitivity analyses, the model application can be tested with the concept of history matching. The concept of history matching is that a model's predictive capability can be shown to reside within acceptable limits by comparing model predictions with a data set independent of the calibration data. If the comparison is unfavorable, the model needs further calibration. If the comparison is favorable, it gives weight to the argument that the model application can be used for prediction with a reasonable assurance of accuracy. This assurance does not, however, extend to conditions other than those tested and thus does not account for unforeseen stresses. History matching shows how the model application can simulate past conditions. It does not necessarily indicate accuracy for predictive simulations.

5-6. Model Execution and Interpretation of Results

a. Model execution. After successfully performing the calibration and sensitivity analysis, the model application is ready for use in performing simulations. This step usually takes less time than the calibration step. Model output is usually produced in the form of hydraulic heads and flow vectors at grid nodes. From these, head contour maps, flow field vector maps, groundwater pathline maps, and water balance calculations can be made using post processors. Some combination of these simulation results can be used to answer the questions posed by the modeling objectives.

b. Dealing with uncertainty. One key issue is how to constrain the modeling runs to account for uncertainty while still meeting the modeling objectives. This can be accomplished by using one of the following approaches:

(1) Best estimate. Producing “best estimate” results by using the most representative input values usually provides a useful indicator of groundwater velocities, heads, and fluxes. However, single value modeling results do not, by themselves, give much assurance of accuracy.

(2) Worst case. One possible modeling objective is to determine if a certain groundwater level or flow rate may arise given the most unfavorable conditions possibly expected. In this case, the model is calibrated using the best estimates from the ranges of input values, but simulations are performed using input values from the most unfavorable end of the input ranges. For example, field estimates for transmissivities are identified at 500-800 m^3/d-m. If model simulations predict that a well field design will meet its production goals even when using transmissivities as low as 400 m^3/d-m, this gives some assurance that the design is adequate.

(3) Best estimate with sensitivity analysis adjustment. Best estimate results can be coupled with the results from the sensitivity analysis to provide a range of expected aquifer performance. For example, if modeling objectives were to predict whether head fluctuations at a location could exceed 5 m, and, if the best estimate results plus additional adjustment from the sensitivity analysis results predict only a total of 2 m expected fluctuation, then further analysis may not be warranted.

(4) Bracketed ranges. In this case, two or more calibrations of the model are made. Each uses a different set of values for key input parameters. The results should bracket the expected possible range of results. For example, if field data defined a dominant hydrogeologic unit at a site, but the only two pumping tests for this unit produced differing estimates of its hydraulic conductivity, a model could be calibrated twice using the two different estimates. These two calibrations would differ in boundary flux and flow velocities which should be checked against observed or estimated values. Bracketing gives those interpreting modeling results a greater understanding of how overall model performance varies according to input uncertainty.

(5) Using uncertainty distributions. Various methods, such as inverse modeling and Monte Carlo analysis, can be used to more fully analyze the effects of uncertainty on modeling results. Usually these methods require the modeler to bound the range of uncertainty or define a probability distribution for the associated variable. Results are returned in bounded or distribution form. The level of effort and computation time required for these types of analyses are much greater than those of earlier described approaches. Advances in software are expected to increase the usability of these approaches in the future.

5-7. Post Audit

A post audit is similar to history matching described in Section 5-5, but differs in that it assesses the accuracy of past predictions compared with data gathered in the interim period (usually over a long period of time). Post audits usually provide insights into model improvements that can be made and weaknesses in making modeling assumptions. Anderson and Woessner (1992) report on four post audits reported in widely available literature none of which accurately predicted the future. They concluded that inaccurate predictions were based on errors in the conceptual models and also on failure to use appropriate values for assumed future stresses. When modeling is viewed as an iterative process continuing over long periods of time, then modeling performed today will provide a basis for future modeling which will be improved by larger data sets and improved technology.

Chapter 6
Interaction Between Surface Water and Groundwater

6-1. General

This chapter will provide an overview of the distribution and movement of water between the surface and the subsurface. Practical analytical methods which quantify the interaction between surface water and groundwater are provided. Additionally, an overview on computer modeling of the interaction of groundwater with surface water is presented.

6-2. System Components

a. General. Surface and groundwater systems are in continuous dynamic interaction. In order to properly understand these systems, the important features in each system must be examined. These features are grouped into components referred to as the surface component, the unsaturated zone component, and the groundwater (saturated) component. The flow of water on the surface, and in the unsaturated and saturated zone, is driven by gradients from high to low potentials. Figure 6-1 presents the basic flow components of a surface-groundwater system.

b. Surface water. Surface water is water that flows directly on top of the ground. Surface water includes obvious features of streams, lakes, and reservoirs, and the less obvious features of sheet flow, and runoff from seeps and springs. Runoff across the surface occurs whenever the accumulation from precipitation (either as rain or snow) exceeds the infiltration capacity of the subsurface strata and the evapotranspiration rate, or whenever the rate of groundwater discharge exceeds that which is evapotranspired.

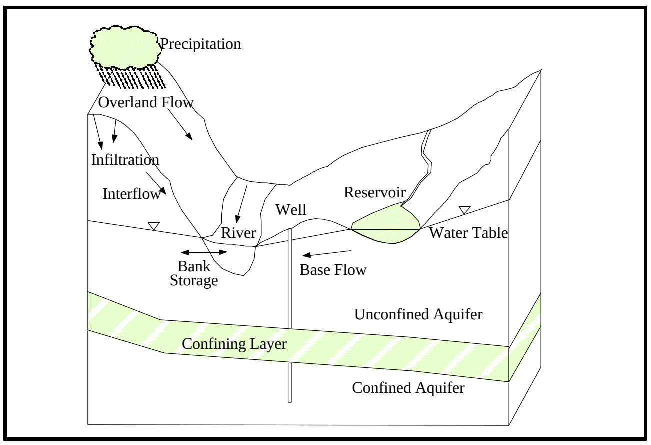

Figure 6-1. Flow components of the surface-groundwater system

c. Subsurface water.

(1) Water infiltrating through the unsaturated zone is from direct precipitation, from overland flow, and from leakage through streambeds. Flow in the unsaturated zone generally is assumed downward in response to gravity. However, poorly permeable strata (for example, a clay layer) can create barriers to downward flow that can limit the amount of water reaching the underlying saturated zone by deflecting flow laterally until it is discharged as evapotranspiration or as seepage to the surface (referred to as interflow).

(2) Groundwater is an important component in the hydrologic cycle as it acts as a large storage reservoir that accepts and releases water from and to the surface. In places where streams flow over permeable strata, the peak flow of a flood is attenuated because of leakage from the stream into the subsurface. Most of this water returns to the stream as the stage decreases, resulting in prolonged flow. This relatively short-term flow of water into and out of the subsurface during a flood event commonly is called bank storage. Additionally, many streams throughout the country have sustained flows during extended dry periods as a result of leakage from groundwater. This contribution to streamflow is called baseflow.

(3) Groundwater flow is complicated by variations in the water-transmitting properties of strata. For example, groundwater can be confined beneath poorly permeable layers of strata (clay or unfractured, dense rock) only to discharge to the surface where land surface cuts through the confining layers or discharge to the surface through fractures that extend through the confining layers (such as a spring). Additionally, pumping of groundwater from permeable sediments near a stream can decrease flow in the stream either by reducing leakage of groundwater to the stream or by inducing leakage from the stream into the subsurface.

6-3. Infiltration

a. General. Infiltration is the process by which water seeps from the surface into the subsurface. The unsaturated zone consists of soil, air, and water (which may be in the form of ice or vapor). The pore space within the soil medium is filled with varying amounts of air and water. Quantifying flow in the unsaturated zone is a much more complicated process than that of the saturated zone primarily because soil properties which control infiltration rates, such as hydraulic conductivity and soil moisture content, tend to change with time.

b. Concepts. Both gravity and moisture potential act to pull water from the surface into the unsaturated zone. Gravity potential is equivalent to elevation (or hydraulic) head. Moisture potential is the negative pressure (or suction) exerted by a soil due to soil-water attraction. The total potential h in unsaturated flow is defined as:

$$h = \Psi(\theta) + Z \qquad (6\text{-}1)$$

where

$\Psi(\theta)$ = moisture potential

Z = gravity potential

Downward flow through the unsaturated zone is controlled by the vertical hydraulic conductivity $K(\theta_v)$ of the soil medium, and moisture potential. The value $K(\theta_v)$ increases as the moisture content increases. At saturation, the vertical hydraulic conductivity $K(\theta_v)$ equals the saturated hydraulic conductivity described by Darcy's Law (Section 2-11), and downward flow is controlled by hydraulic conductivity and elevation head. Moisture potential varies with the moisture content and pore size of the medium. When soil is dry, the moisture potential is typically several orders of magnitude greater than gravity potential. A dry soil will have a higher initial infiltration rate than that of a moist soil, due primarily to free surfaces within the pore space. The pores act as capillary tubes to draw in water, and as they fill, the capillary forces decrease along with the infiltration rate. Figure 6-2 provides an illustration of the relationship between moisture potential, hydraulic conductivity, and water content for a clay sample.

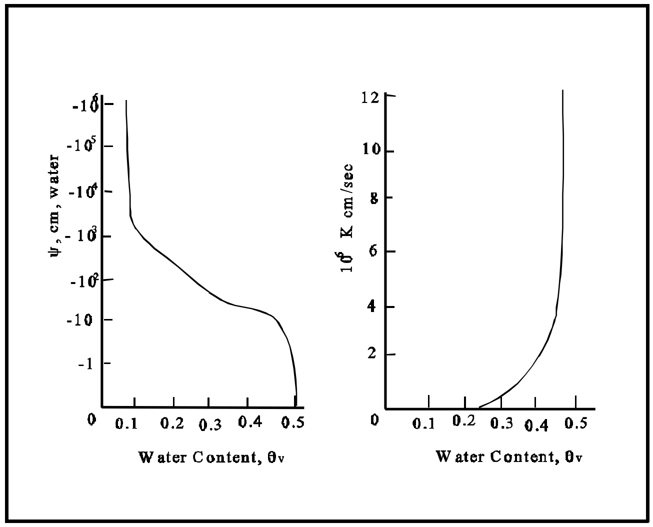

Figure 6-2. Typical relationship between moisture potential (ψ), hydraulic conductivity (*K*) and water content (θ_v) for an identical clay sample

Groundwater flow is described by Darcy's Law:

$$q = -K\,(dh/dl) \qquad (6\text{-}2)$$

where

q = flow rate of groundwater

K = hydraulic conductivity

dh/dl = gradient over which flow occurs

By substituting ψ+z in Equation 6-1 for h in Equation 6-2, infiltration can be described as a function of vertical hydraulic conductivity and moisture potential:

$$q_v = -K_v \frac{\partial(\psi + z)}{\partial z} \qquad (6\text{-}3)$$

As discussed earlier, infiltration and flow in the unsaturated zone are controlled by moisture potential $\psi(\theta)$ as well as hydraulic conductivity $K(\theta_v)$. Thus, any change in the infiltration rate requires a change in moisture content θ. This is described by the one-dimensional Richard's equation:

$$\frac{\partial}{\partial z}\left[K(\theta_v)\left(\frac{\partial\psi}{\partial z} + \frac{\partial z}{\partial z}\right)\right] = \frac{\partial\theta}{\partial t} \qquad (6\text{-}4)$$

The solution to Richard's equation indicates a decrease in moisture potential with cumulative infiltration, and as moisture potential approaches zero, the infiltration rate decreases to a rate equivalent to saturated vertical hydraulic conductivity; i.e.,

$$q \text{ (at saturation)} = -K_v\, (dz/dz) = -K_v \qquad (6\text{-}5)$$

Infiltrating moisture from rainfall events tends to move vertically downward as a wave front of saturated soil. Eventually, this wave front reaches the water table and moisture conditions in the soil profile stabilize and return to their pre-rain state.

c. Infiltration capacity curve. The decrease in moisture potential with cumulative infiltration is illustrated by an infiltration capacity curve (Figure 6-3). The initial (or antecedent) infiltration capacity f_0 is typically controlled by the moisture content of the soil. The final (or equilibrium) infiltration capacity f_c is equivalent to the saturated vertical hydraulic conductivity of the soil K_v.

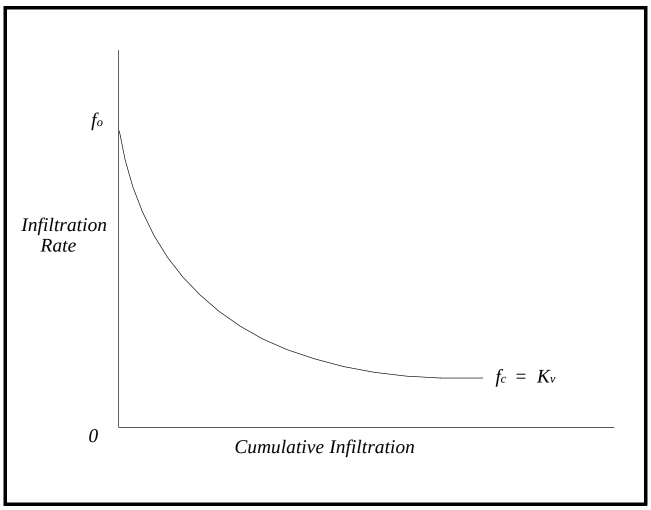

Figure 6-3. Infiltration capacity curve

Other factors affecting soil infiltration include: rainwater chemistry, soil chemistry, organic matter content, and presence of roots and burrowing animals. Field conditions that encourage a high infiltration rate include: low soil moisture, course/porous topsoil, well-vegetated land (inhibited overland flow, lower soil moisture due to transpiration), and land use practices that reduce soil compaction.

6-4. Stream-Aquifer Interaction

Water is transmitted from a basin to a channel system through the following three basic mechanisms (Figure 6-1):

- Overland flow occurs when the rate of precipitation exceeds the infiltration capacity of the local soil.

- Interflow occurs when infiltrating subsurface flow above the water table is diverted towards a channel bed by stratigraphic changes.

- Base flow occurs when the potentiometric surface (or elevation of the water table) proximate to the channel bed exceeds stream elevation.

Seasonal conditions may control the groundwater elevation and thus the direction of flow between the stream and aquifer. When the hydraulic gradient of the aquifer is towards the stream, groundwater discharges to the stream, and the stream is a gaining or effluent stream. When the hydraulic gradient of the aquifer is away from the stream, the stream is losing or influent. The rate of this water loss is a function of the depth of water, the hydraulic gradient towards the groundwater, and the hydraulic conductivity of the underlying alluvium. The channel system can be hydraulically connected to the aquifer, or have a leaking bed through which water can infiltrate to the subsurface. The extent of this interaction depends on physical characteristics of the channel system such as cross section and bed composition. Streams commonly contain a silt layer in their beds which reduces conductance between the stream and the aquifer.

6-5. Interaction Between Lakes and Groundwater

The hydrologic regime of a lake is strongly influenced by the regional groundwater flow system in which it is located. This interaction plays a critical role in evaluating the water budget for the lake. A method of classifying lakes hydrogeologically can be based on the domination of the annual water budget on surface water or groundwater. Lakes dominated by surface water typically have inflow and outflow streams, while seepage lakes are groundwater dominated. Large permanent lakes almost always provide areas of discharge from the local groundwater. The rates of groundwater inflow are controlled by watershed topography and the hydrogeologic environment. Winter (1976) concluded that if the water table is higher than the lake level on all sides of a seepage lake, groundwater will seep into the lake from all sides, including upward seepage through the lake bottom assuming a homogeneous flow system. However, should an aquifer of much higher conductivity underlie the lake, this zone of upward seepage can be eliminated. Three-dimensional numerical analysis of the lake/groundwater interaction system indicated that upward seepage tends to occur around the lake edges, while seepage out of the lake tends to occur in the middle of the lake.

6-6. Analytical Methods

a. General. This section will provide physically based analytical methods for:

(1) Estimating aquifer diffusivity (Section 2-16) from the response of groundwater levels to fluctuations in surface-water levels.

(2) Estimating the groundwater contribution of recharge from a storm event to streamflow.

(3) Using streamflow records to estimate aquifer diffusivity.

(4) Estimating the effects of pumping wells on stream depletion.

b. Baseflow recession. A stream hydrograph describes the flow at a certain point on a river as a function of time. While the overall streamflow shown on a hydrograph gives no indication of its origin, it is possible to break down the hydrograph into components such as overland flow, interflow, and baseflow. After a critical time following a precipitation event when overland flow and interflow are no longer contributing to streamflow, the hydrograph of a stream will typically decay exponentially. Discharge during this decay period is composed entirely of groundwater contributions as the stream drains water from the declining groundwater reservoir. This baseflow recession for a drainage basin is a function of the overall topography, drainage patterns, soils, and geology of the watershed. The slope of baseflow recession is consistent for each watershed and independent of such things as magnitude of the precipitation event or peak flow. When an aquifer contained by a watershed is homogeneous, the hydrograph of a stream at a critical time following a precipitation event (when all discharge to the stream is contributed by groundwater) will decay following an exponential curve. This baseflow recession is described by:

$$Q = Q_0 e^{-kt} \tag{6-6}$$

where

Q = flow at some time t after recession has started

Q_0 = flow at the start of baseflow recession

k = recession constant for the basin

t = time

The value for k, the recession constant, is typically estimated empirically from continuous hydrograph records over an extended period. Rorabaugh (1964) developed a physically based method for estimating the recession constant for the basin based upon aquifer diffusivity (Section 2-17) and basin topography. This allowed for the estimation of baseflow recession in streams with limited continuous data, and also allowed for the estimation of adjacent groundwater properties based upon measured streamflow records.

c. Assumptions. To analyze a stream-aquifer system analytically, many simplifying assumptions need to be made. Assumptions used throughout Sections 6-7, 6-8, and 6-9 include the following:

(1) Darcy's law applies.

(2) The aquifer is homogenous, isotropic, and of uniform thickness.

(3) The rocks beneath the aquifer are impermeable.

(4) The surface-water body fully penetrates the groundwater system, and flow is considered horizontal.

(5) The lateral boundaries of the aquifer are impermeable.

(6) Distances from the stream to groundwater divides or geologic boundaries of flow are for each stream reach; when this distance is termed semi-infinite, this boundary has minimal influence on the analytical solution.

(7) The river is not separated from the aquifer by any confining material.

6-7. Estimating the Transient Effects of Flood Waves on Groundwater Flow

a. Introduction. Accurate estimation of the transmissivity and storage coefficient of an aquifer is critical to the prediction of groundwater flow patterns. If an aquifer is adjacent to a river or surface reservoir which experiences periodic stage fluctuations, it may be possible to calculate these parameters from an analysis of the aquifer response to the fluctuations.

b. Principle. The idealized flow domain is shown in Figure 6-4. The aquifer is represented as a semi-infinite, horizontal confined aquifer of uniform thickness bounded on the left by a reservoir (open boundary). The surface-water body is assumed to completely penetrate the aquifer. The water level in the reservoir fluctuates and causes a corresponding fluctuation in the piezometric head within the aquifer. The one-dimensional flow system is described by the governing equation for linear, non-steady flow in a confined aquifer (see Equation 2-28):

$$\frac{\partial h}{\partial t} = \frac{T}{S}\frac{\partial^2 h}{\partial x^2} \qquad (6\text{-}7)$$

where

h = rise or fall of piezometric head in the aquifer [L]

x = distance from aquifer-surface body intersection [L]

t = time [T]

S = aquifer storage coefficient

T = aquifer transmissivity [L^2/T]

T/S = aquifer diffusivity [L^2/T]

c. Review of solutions. The solution to the governing equation (Equation 6-7) subject to a fluctuating boundary condition has been presented by several authors (Ferris 1951; Cooper and Rorabaugh 1963; Pinder, Bredehoeft, and Cooper 1969; Hall and Moench 1972). Each of the solutions was derived for the semi-infinite flow domain described above subject to the assumptions listed in Section 6-6. The solutions are derived for confined conditions, although satisfactory results for unconfined conditions will be obtained if:

(1) The location of the computed head is sufficiently far enough from the surface water intersection so that it is unaffected by vertical components of flow.

(2) The range in cyclic fluctuation at the computed location is only a small fraction of the saturated thickness of the formation.

d. Uniform fluctuations. Ferris (1951) observed that wells near bodies of tidal water often exhibit sinusoidal fluctuations of water level in response to periodic changes in tidewater stage. An analogous response was suggested for wells situated adjacent to large surface-water bodies. When the stage of the surface body fluctuates as a simple harmonic motion, a series of sinusoidal waves is propagated outward from the surface body-aquifer intersection through the aquifer. Expressions were developed to determine aquifer diffusivity (T/S) based on the observed values of amplitude, lag, velocity, and wavelength of the sinusoidal changes in groundwater level. If the range of the fluctuation in surface water and an adjacent well is known, aquifer parameters can be derived by:

$$h_{gw} = 2H_{sw}e^{-d\sqrt{\frac{\pi S}{PT}}} \qquad (6\text{-}8)$$

If the lagtime in occurrence between surface and groundwater maximum or minimum stages is known, then:

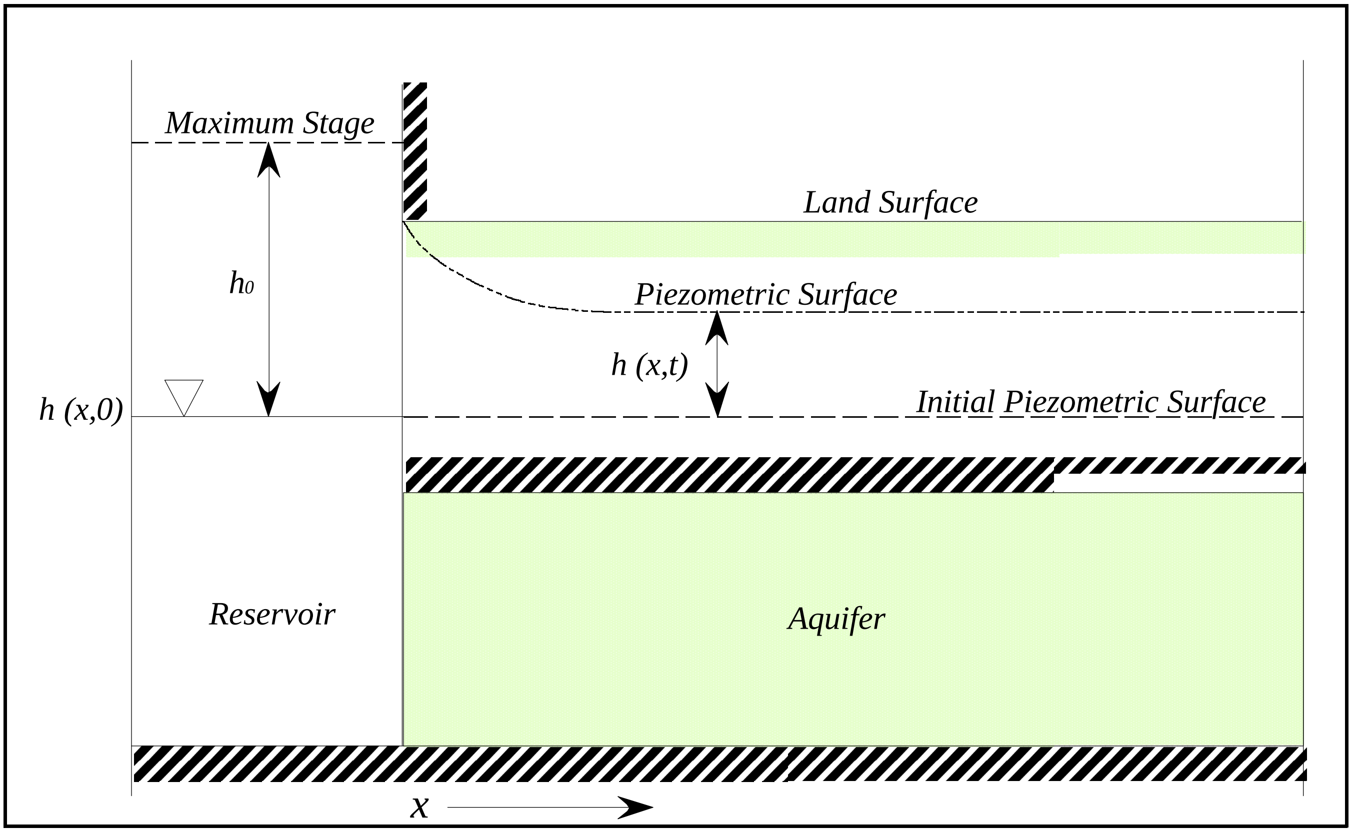

Figure 6-4. Representation of simplified one-dimensional flow as a function of surface-water stage

$$t_{lag} = d\sqrt{\frac{PS}{4\pi T}} \tag{6-9}$$

where

h_{gw} = maximum rise in groundwater

H_{sw} = maximum rise in surface-water body

d = distance from well to surface water

S = aquifer storage coefficient

T = aquifer transmissivity

P = period of uniform tide or stage fluctuation

t_{lag} = lag time in occurrence of maximum groundwater stage following the occurrence of a similar surface stage

As shapes of the stage hydrographs for flood waves in surface streams and reservoirs vary widely, a solution of the governing equation satisfying a boundary condition described by a uniform sine wave will generally not approximate the actual domain adequately.

e. Example problem. Sunny Bay has tidal fluctuations every 12 hr with a total tidal change of 3 m. A screened monitoring well 200 m from the shoreline is located within a confined aquifer that is 10m thick. The amplitude of the groundwater change due to the tides is 1 m. S was estimated to be 0.001. Estimate the hydraulic conductivity K of the aquifer.

Given

h_{gw} = 1 m

H_{sw} = 3 m

d = 200 m

$S = 0.001$

$b = 10$ m

$P = 12$ hr (0.5 day)

Determine K by using Equation 6-8, and substituting Kb for T (Equation 2-9).

$$h_{gw} = 2H_{sw}e^{-d\sqrt{\frac{\pi S}{PT}}}$$

$$1\text{m} = 2(3\ \text{m})\, e^{-200\ \text{m}\sqrt{\frac{\pi(0.001)}{(0.5\ \text{days})(10\ \text{m})(K)}}}$$

$$\ln 0.1667 = \ln(e^{-200\ \text{m}\sqrt{\frac{\pi(0.001)}{(0.5\ \text{days})(10\ \text{m})(K)}}})$$

$$-1.79 = -200\ \text{m}\sqrt{\frac{\pi(0.001)}{(0.5\ \text{days})(10\ \text{m})(K)}}$$

$$0.0090\ \text{m}^{-1} = \sqrt{\frac{0.0006\ \text{m}^{-1}\text{days}^{-1}}{K}}$$

$$0.0001\ \text{m}^{-2} = \frac{0.0006\ \text{m}^{-1}\text{days}^{-1}}{K}$$

$$K = 6\ \text{ft/day}$$

f. Representation of fluctuations by discrete steps. Pinder, Bredehoeft, and Cooper (1969) developed solutions to the governing equation using a discrete approximation of the surface body stage hydrograph. This discrete approach allows the use of a stage hydrograph of any shape (i.e., one not restricted to sinusoidal or uniform asymmetric curves). For each increment in reservoir stage, the head in the adjacent semi-infinite aquifer is given by the solution of Equation 6-7 subject to the boundary and initial conditions

$$h(0,t) = \begin{cases} 0 & when\ t \le 0 \\ \Delta H_m & when\ t > 0 \end{cases} \tag{6-10}$$

$$h(\infty,t) = 0 \tag{6-11}$$

$$h(x,0) = 0 \qquad when\ x \ge 0 \tag{6-12}$$

where

Δh_m = instantaneous rise in surface-water stage at time $t = m\Delta t$ where m is an integer

The solution to the problem is given by:

$$\Delta h_m = \Delta H_m erfc \frac{x}{2\sqrt{(T/S)t}} \tag{6-13}$$

The complementary error function (*erfc*) is unique for each value of x:

$$erfc(x) = \int_x^\infty e^{-t^2} dt \tag{6-14}$$

Values of *erfc* can be found in tables from many sources.

To compute the change in aquifer head (h_p) at the end of any number of stage increments, the change in surface-water stage ΔH_m after each successive increment time Δt must be obtained. The change in groundwater head is given by summing the values of Δh_m computed for each ΔH_m over the period $(p\text{-}m)\Delta t$, giving:

$$h_p = \sum_{m=1}^{p} \Delta H_m \left\{ erfc \frac{u}{2\sqrt{p-m}} \right\} \tag{6-15}$$

where

h_p = head at a distance x from the reservoir intersection at time $p\Delta t$;

$p\Delta t$ = total time since beginning the period of analysis, where p is the number of time intervals.

$$u = \frac{x}{\sqrt{(T/S)\Delta t}} \tag{6-16}$$

Equation 6-16 can be used to generate type curves for different values of diffusivity. Each set of curves therefore represents the computed change in hydraulic head due to a change in the surface-water stage when selected diffusivities are assumed. The diffusivity of the aquifer is then obtained by choosing from the set of type curves the one which best matches the response observed in adjacent observation wells.

6-8. Estimating Baseflow Contribution from Storm Events to Streamflow

a. Instantaneous recharge.

(1) For flood event scenarios where the precipitation event is of short duration, the assumption of instantaneous recharge can often be made. Rorabaugh (1964) derives an equation which describes base-flow recession at a critical time after an instantaneous uniform increment of recharge ceases to calculate groundwater discharge to a stream:

$$q = 2T(\frac{h_0}{a})(e^{-\pi^2 Tt/4a^2S} + e^{-9\pi^2 Tt/4a^2S} + ...) \quad (6\text{-}17)$$

where

q = groundwater discharge [cfs] per foot of stream length (one side) at any time

t = time [days] after recharge ceases [L/T]

h_0 = an instantaneous water table rise, in feet [L]

T = aquifer transmissivity, in ft^2/day [L^2/T]

S = storage coefficient [dimensionless], a common estimate for this value in an alluvial aquifer is 0.20

a = distance from stream to groundwater divide, in feet [L]

(2) This relationship assumes the initial condition that groundwater levels are equal to stream level, and water table fluctuations are small compared to total aquifer thickness.

(3) When $Tt/a^2S > 0.2$, the terms in the series of Equation 6-17 become very small and may be neglected (Rorabaugh 1964):

$$q = 2T(\frac{h_0}{a})e^{-\pi^2 Tt/4a^2S} \quad (6\text{-}18)$$

Conversely, when $Tt/a^2S < 0.2$, Equation 6-17 can be estimated as (Rorabaugh 1964):

$$q = h_0\sqrt{\frac{ST}{t\pi}} \quad (6\text{-}19)$$

(4) Equation 6-19 is for time sufficiently small so that the aquifer response has not reached the groundwater divide, and therefore may be applied to semi-infinite conditions.

(5) The term 'critical time' is defined as the time required in a recession for the profile shape to stabilize, allowing for a straight plot on semi-log graph of streamflow versus time (water levels fall exponentially with time). From stream records, the logarithmic slope of the baseflow recession can be derived after critical time. Critical time (t_c) defines the point on a flow hydrograph where water moving into a stream following a recharge event is derived solely from groundwater (i.e., overland flow in the watershed is no longer a component). Mathematically this term was defined by integrating Equation 6-17 with respect to time:

$$t_c = \frac{0.2a^2S}{T} \quad (6\text{-}20)$$

(6) The total groundwater remaining in storage V from the recharge event, which will eventually be transmitted to a stream at any time after t_c along an entire stream reach l, can be estimated by:

$$V = 2ql(\frac{4a^2S}{\pi^2 T}) \quad (6\text{-}21)$$

b. Constant rate of recharge. For the case of a constant rate of recharge (or constant rate of change in river stage), Rorabaugh (1964) derived the following equation:

$$q = CaS\left[1 - \frac{8}{\pi^2}\left(e^{-\pi^2 Tt/4a^2S} + \frac{1}{9}e^{-9\pi^2 Tt/4a^2S} + \frac{1}{25}e^{-25\pi^2 Tt/4a^2S} + \ldots\right)\right] \tag{6-22}$$

where

$C = dh/dt$, which is the rate of rise of the water table associated with constant recharge

For values of $Tt/2aS > 2.5$, the exponential terms become insignificant, and flow approaches the steady-state condition:

$$q = CaS \tag{6-23}$$

For early time when $Tt/a^2S < 0.2$, effects will not have reached the boundary and the flow is the same as that for a semi-infinite case:

$$q = C(2/\sqrt{\pi})\sqrt{TSt} \tag{6-24}$$

c. Constant rate of recharge over a specified time. For the case of constant rate recharge beginning at time t=0, and stopping at time t', Rorabaugh (1964) derived the following equation:

$$q = CaS(8/\pi^2)\left[e^{-\pi^2 Tt'/4a^2S} - e^{-\pi^2 Tt/4a^2S} + \frac{1}{9}e^{-9\pi^2 Tt'/4a^2S} - \frac{1}{9}e^{-9\pi^2 Tt/4a^2S} + \ldots\right] \tag{6-25}$$

Analytical methods presented in Section 6-8 assume that precipitation instantaneously recharges the water table. Thus, flow in the unsaturated zone is not addressed. A more accurate estimation of baseflow contribution to streamflow from a storm event can be derived by accounting for soil moisture conditions in the unsaturated zone. However, this accounting requires a numerical complexity that can only be addressed by computer models.

6-9. Estimating Aquifer Diffusivity from Streamflow Records

a. Theory. When critical time is reached, the recession curve on a semilog graph becomes a straight line. Rorabaugh (1960) developed an equation for estimating aquifer diffusivity (T/S) from the slope of the water level recession in a stream or observation well after critical time:

$$\frac{T}{S} = \frac{0.933a^2\log(h_1/h_2)}{(t_2 - t_1)} \tag{6-26}$$

where

a = distance from stream to groundwater divide [L]

T = aquifer transmissivity [L^2/T]

S = storage coefficient [dimensionless]

h_1 = initial water level [L], at time t_1 [T]

h_2 = water level [L], at time t_2 [T]

This equation is applicable for the condition where recharge is instantaneous and evenly distributed. Assumptions in addition to those stated in Section 6-6 include that the aquifer is thick relative to the change in water level and that the aquifer is wide relative to the thickness. If the base-flow recession curve is evaluated after critical time to determine the time required for streamflow to decline through one log cycle (Δt/log cycle), Equation 6-26 reduces to:

$$\frac{a^2S}{T} = \frac{\Delta t/\text{log cycle}}{0.933} \tag{6-27}$$

Combining Equations 6-20 and 6-27 yields a critical time of:

$$t_c = \frac{0.2(\Delta t/\text{log cycle})}{0.933} \tag{6-28}$$

These equations also allow for the estimation of baseflow recession if average values of aquifer diffusivity and the distance from the stream to the groundwater divide can be estimated.

b. Methodology. To determine T/a^2S from a recession curve which is declining exponentially with time, the following procedure can be followed (Bevans 1986):

(1) The time that recharge occurred is assumed to be at the point at which streamflow hydrographs reach their peak.

(2) The slope of the baseflow recession curve (days/log cycle) is determined from the recession curve after it becomes a straight line, either by the observed decrease through one log cycle, or by extrapolating the straight line part of the baseflow recession curve through one log cycle.

(3) The slope of the base-flow recession curve is inserted into Equation 6-28 as Δt/log cycle, and the critical time t_c (days) is computed.

(4) The computed critical time is checked against the streamflow hydrograph. The computed critical time needs to be equivalent to the period from the hydrograph peak to the point on the recession curve where the curve becomes a straight line. If computed and observed critical times are the same, then the slope of the baseflow recession curve can be used in Equation 6-27 to compute T/a^2S (1/days). If the computed and observed critical times differ significantly, then extraneous factors probably are affecting the slope of the baseflow recession curve and that particular streamflow record is not appropriate for determining T/a^2S.

(5) T/a^2S values should be determined from several baseflow recession curves, representing different ranges of baseflow rate, and compared to see if T/a^2S is constant. If the values are constant within a narrow range, then T/a^2S can be considered a stream-aquifer constant. Once aquifer diffusivity (T/S) is estimated, the value of transmissivity can be estimated by approximating S from tables; i.e., $S \approx 0.20$ for typical sand aquifers.

c. Example problem.

Given: Extrapolated slope of baseflow recession equals 32 days/log cycle (Figure 6-5). The aquifer is unconfined and consists of shallow sands underlain by bedrock. Assume the aquifer storage coefficient equals 0.20. The stream is located in a valley bounded by bedrock approximately 1,000 m on each side of the stream. Estimate aquifer transmissivity.

Solution:

$$\frac{a^2S}{T} = \frac{\Delta t/\text{log cycle}}{0.933}; \quad \frac{(1{,}000\ \text{m})^2(0.20)}{T}$$

$$= \frac{32\ \text{days}}{0.933}$$

$$T = 5{,}800\ \text{m}^2/\text{day}$$

6-10. Estimating Effects of Pumping Wells on Stream Depletion

a. Assumptions. Jenkins (1968) created a series of dimensionless curves and tables which can be applied to a stream-aquifer system under the following assumptions:

(1) The aquifer is isotropic, homogeneous, and semi-infinite.

(2) Transmissivity remains constant.

(3) The stream is of constant temperature, represents a straight boundary, and fully penetrates the aquifer.

(4) Water is released instantaneously from storage.

(5) The pumping rate is steady during any rate of pumping.

(6) The well is screened through the full saturated thickness of the aquifer.

b. Applications. Computations can be made of:

(1) The rate of stream depletion at any time during the pumping period or the following non-pumping period.

(2) The volume of water induced from the stream during any period, pumping or non-pumping.

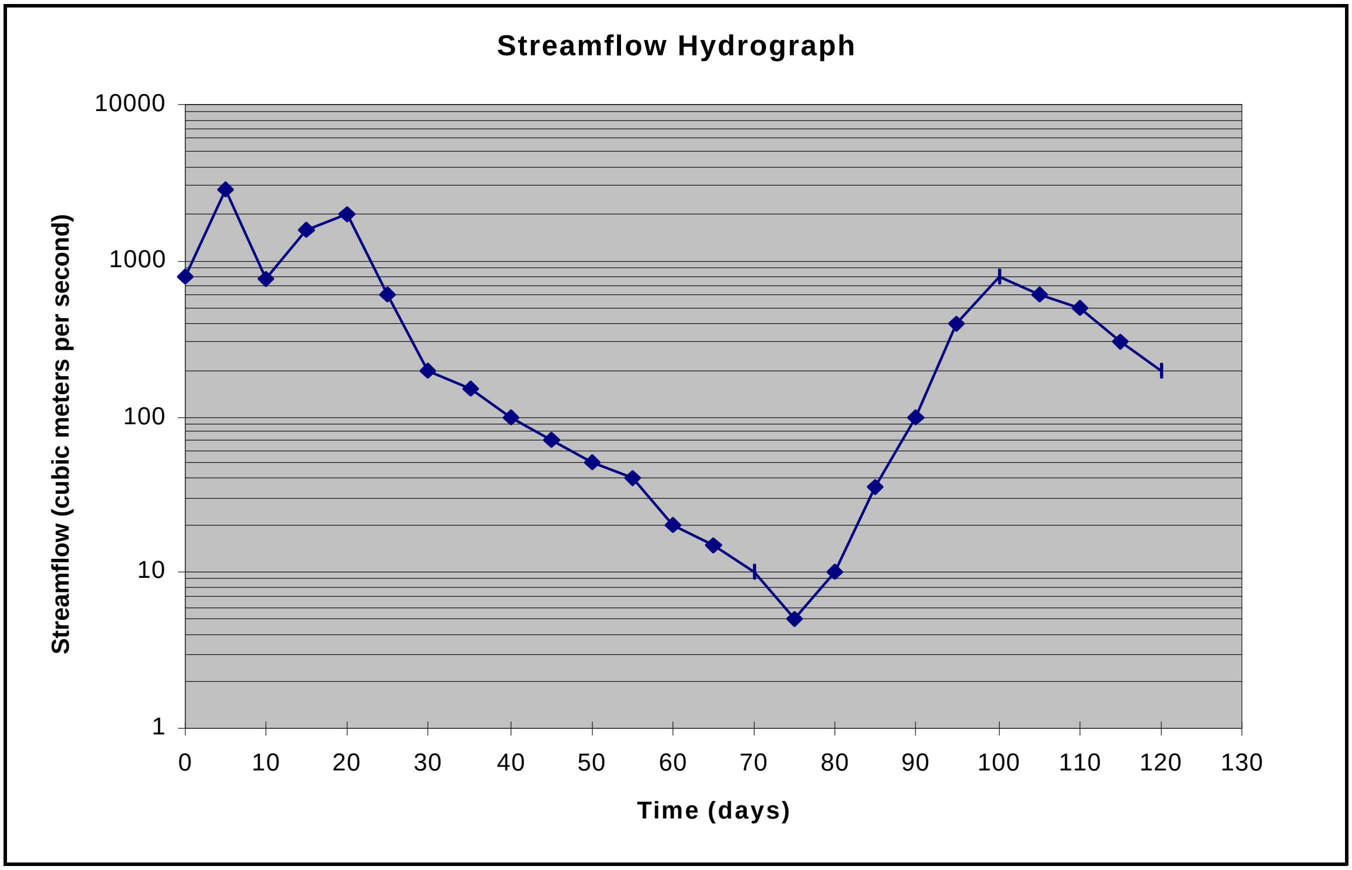

Figure 6-5. Hypothetical streamflow hydrograph

(3) The effects, both in volume and rate of stream depletion, of any pattern of intermittent pumping (Jenkins 1968).

c. Stream depletion factor. Stream depletion means either direct depletion of the stream or reduction of groundwater flow to the stream. In his report, Jenkins introduces a 'stream depletion factor' (sdf) term. If the system meets the above assumptions:

$$sdf = a_w^2 S/T \tag{6-29}$$

where

a_w = distance from the stream to the pumping well

In a complex system, *sdf* can be considered an effective value of $a_w^2 S/T$. This value is dependent upon the integrated effects of irregular impermeable boundaries, stream meanders, areal variation of aquifer properties, distance from the stream, and imperfect connections between the stream and aquifer.

d. Methodology. A simple application of determining the effects of a well, located a given distance *a* from a stream, pumping at a constant rate *Q* for a given time *t* on the volume of stream depletion *v* can be derived from Figure 6-6. First compute the value of *sdf*, then estimate the ratio of *v/Qt* from Figure 6-6. Additionally, the effects of pumping on the rate of stream depletion at a given time *t* after pumping commenced can be easily determined by using Figure 6-6 to determine the ratio *q/Q*. Conversely, the time after pumping begins in which stream depletion will equal a predetermined percentage of the pumping rate can be determined by first computing the ratio of *t/sdf*. Computations for estimating the effects on the river after pumping has stopped, intermittent pumping, and the volume of water induced from the stream during any pumping or non-pumping period can be derived from additional charts and tables in the Jenkins (1968) report.

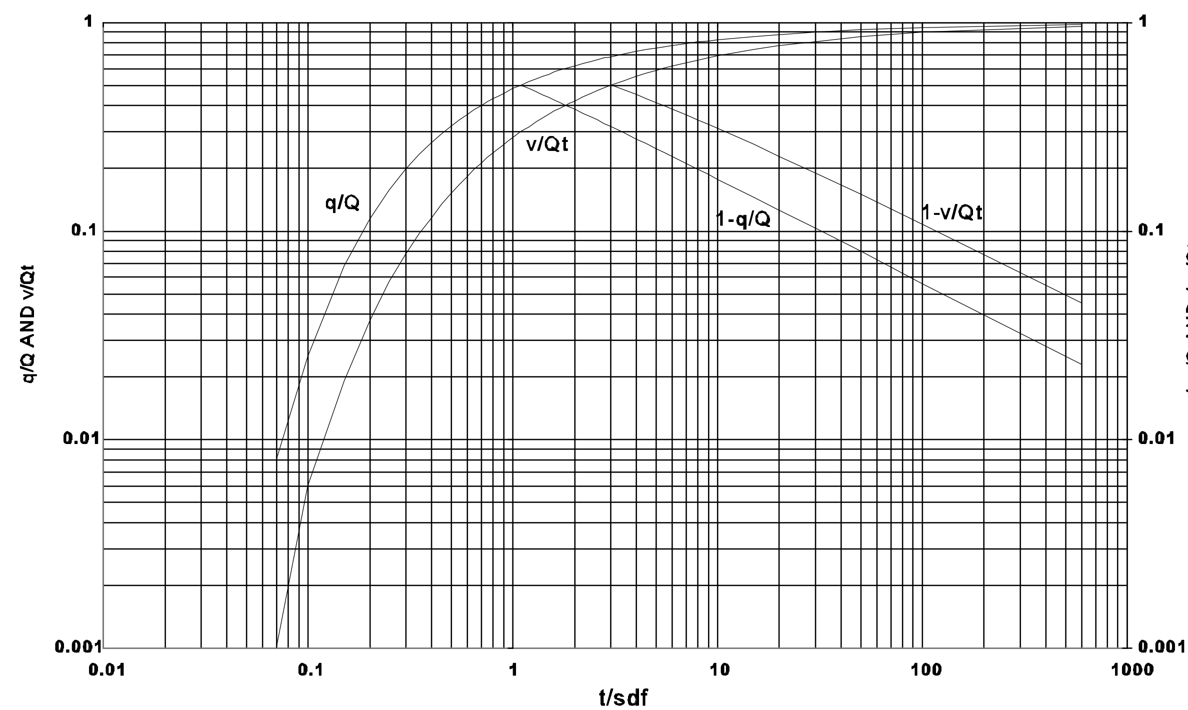

Figure 6-6. Curves to determine rate and volume of stream depletion (Jenkins 1968)

e. Example problem.

(1) The potential influence of aquifer pumping on an effluent stream must be determined. The depth of the stream is 10 m, and depth of the aquifer is 30 m. Assume the stream fully penetrates the aquifer. The hydraulic conductivity of the aquifer is 50 m/day, and specific yield is 0.25. A recently constructed well is located 500 m from the stream (Figure 6-7) and begins pumping at a rate of 1,000 m^3/day.

(2) How much is the total volume of flow in the stream reduced by the pumping well after 2 weeks of pumping?

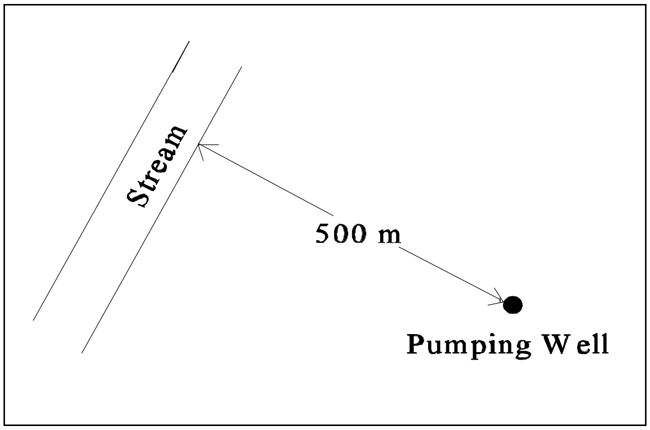

Figure 6-7. Hypothetical stream and pumping well

(a) Given:

Unconfined aquifer
K = 50 m/day
b = 30 m
Q = 1,000 m^3/day
a = 500 m
S = 0.25

(b) Find transmissivity:

$T = Kb$ = (50 m/day)(30 m) = 1,500 m^2/day

(c) Find stream depletion factor (*sdf*):

$sdf = a^2S/T$ = (500 m)^2 (0.25)/(1,500 m^2/day) = 41.7 days

(d) Estimate ratio of v/Qt from Figure 6-6:

t/sdf = 14 days/41.7 days = 0.33

This gives a v/Qt value of approximately 0.09

(e) Solve for v (total stream depletion)

v = (Q)(t)(0.09) = (1,000 m^3)(14 days)(0.09)
v = 1,260 m^3

(3) What is the rate of stream depletion after 2 weeks of pumping?

(a) The rate of streamflow depletion can be solved by:

t/sdf = 0.3, q/Q = 0.22

(b) q = (0.22)(Q). The streamflow is depleted by a rate of 1/5 the pumping rate (220 m^3/day) after 2 weeks of pumping.

(c) Therefore, the total flow in the stream will be depleted by a total of 1,260 m^3 during the first 2 weeks the well is pumping.

6-11. Numerical Modeling of Surface Water and Groundwater Systems

a. General. Although mathematically exact, analytical models generally can be applied only to simple one-dimensional problems because of rigid boundary conditions and simplifying assumptions. However, for many studies, analysis of one-dimensional flow is not adequate. Complex systems do not lend themselves to analytical solutions, particularly if the types of stresses acting on the system change with time. Numerical models allow for the approximation of more complex equations and can be applied to more complicated problems without many of the simplifying assumptions required for analytical solutions. Computer simulation of the interrelationships between surface water and groundwater systems requires the mathematical description of transient effects on potentially complex water table configurations. Ideally, a computer model of the surface-water/groundwater regime should be able to simulate three-dimensional variable-saturated flow including: fluctuations in the stage of the surface-water body, infiltration, flow in the unsaturated zone, and flow in the saturated zone. Additionally, simulation of watershed runoff, surface-water flow routing, and evapotranspiration will allow for completeness. However, this is often a complex task, and no matter how powerful the computer or sophisticated the model, simplifying assumptions are necessary.

b. Modeling stream-aquifer interaction.

(1) General. Numerical models provide the most powerful tools for analysis of the surface-water/groundwater regime. Commonly, interaction between surface water and groundwater is only addressed in the most rudimentary terms. The perspective of the model is of primary importance. In surface-water models, the interaction between surface water and groundwater is often represented as a “black box” source/sink term. Conversely, in groundwater models, surface water is often represented as an infinite source of water, regardless of availability. However, a more precise simulation of the impacts of this interaction can be necessary depending on the objectives of the modeling study.

(2) Theory. From the groundwater perspective, a common simplifying assumption made to ease numerical simulation is that simulation of unsaturated flow is not addressed, and leakage from surface water to an aquifer is assumed to be instantaneous; i.e., no head loss occurs in the unsaturated zone. This assumption is usually reasonable in the common situation where the thickness of the unsaturated zone between the stream and aquifer

is not large. The interaction between surface water and the underlying aquifer can be represented by the partial differential equation of groundwater flow (Equation 2-20):

$$\frac{\partial}{\partial x}\left(K_{xx}\frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial y}\left(K_{yy}\frac{\partial h}{\partial y}\right) + \frac{\partial}{\partial z}\left(K_{zz}\frac{\partial h}{\partial z}\right) - W = S_s\frac{\partial h}{\partial t} \tag{6-30}$$

where

W = flow rate per unit volume of water added to or taken from the groundwater system

Most, but not all, interaction between groundwater and surface water is lumped into the "W" term.

(3) Specified head and specified flux boundaries. The simplest approach in modeling stream-aquifer interaction is to represent surface water as a specified (constant) head or a specified (constant) flux boundary within the groundwater model grid (Section 5-3). In the case of a specified head boundary, the head at the surface-water location is specified as the elevation of water surface. The flow rate to or from the boundary is computed from heads at adjacent grid points using Darcy's law. This type of boundary does not require a 'W' term in the partial differential equation of groundwater flow. For the case of the constant, or specified flux boundary, the flow rate is specified in the model grid as a "known" value of recharge or discharge, and the model computes the corresponding head value through the application of Darcy's law. This type of boundary requires the "W" term in the partial differential equation of groundwater flow.

A major disadvantage to specified head and flux stream boundary representations is that they do not allow for a lower hydraulic conductivity across the seepage interface, or account for the elevation of the streambed bottom. Thus, leakance from the river continues to increase as the water table drops below the streambed.

(4) Head-dependent flux boundary. A second approach is to represent the stream as a head-dependent flux boundary. A head-dependent flux boundary is a common type of value-dependent boundary discussed in Section 5-3. In this type of boundary condition, the flow which is computed at the stream-aquifer interface is computed as a function of the relative water levels for each stress period. This functional relationship is both included in the "W" term of the partial differential equation of groundwater flow and is typically derived from Darcy's law. Thus, the value of groundwater head occurs in the "W" term, and in the space derivatives, which can add difficulty to the solution.

(a) Most groundwater flow models incorporate one or more functions built in to handle this functional relationship. For stream-aquifer relationships, this can be represented as:

$$Q = CRIV(h_{riv} - h_{gw}) \tag{6-31}$$

where

Q = flow between the stream and the aquifer

$CRIV$ = streambed conductance

h_{riv} = river stage

h_{gw} = groundwater elevation

The streambed conductance term represents the product of hydraulic conductivity K and cross-sectional area of flow LW divided by the length of the flow path M:

$$CRIV = KLW/M \tag{6-32}$$

Formulation of a streambed conductance term is illustrated in Figure 6-8. If reliable field measurements of stream seepage are available, they may be used to calculate effective conductance. Otherwise, a conductance value must be chosen more or less arbitrarily and adjusted during model calibration. Values for the cross-sectional area of flow can typically be utilized to guide the initial choice of conductance. In general, however, it should be recognized that formulation of a single conductance term to account for three-dimensional flow processes is inherently an empirical exercise, and that adjustment during calibration is almost always required (McDonald and Harbaugh 1988).

(b) An important assumption common in head-dependent flux boundaries of stream-aquifer

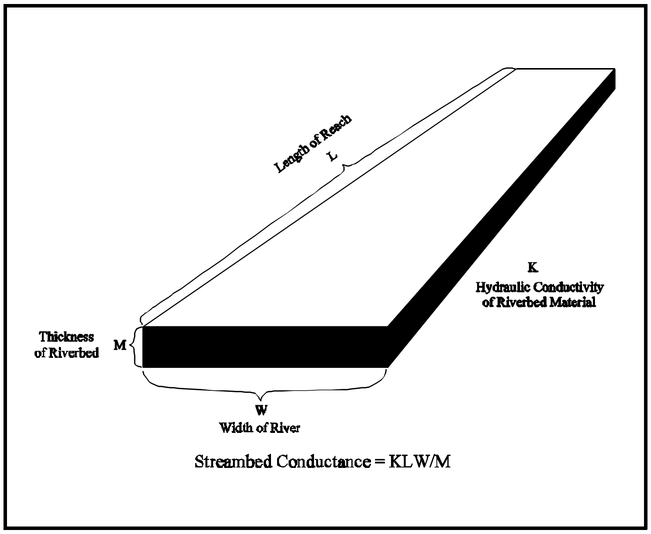

Figure 6-8. Determination of streambed conductance

relationships is that the head differential between the stream and the aquifer is never greater than the sum of stream depth and streambed thickness. In other words, the value of leakage to groundwater does not increase as the groundwater elevation drops below the streambed, and recharge is instantaneous to groundwater. The value of stream bottom elevation is thus also entered into the computational process.

(5) Relationship between cell size and stream width. In groundwater modeling, the smallest unit of homogeneity is represented by the grid cell. Thus, flow between a stream and aquifer is distributed equally over the area of the cell face. For example, if a stream has a width of 50 ft, and the model cell has a width of 300 ft, the same total flow between the stream and the aquifer will be distributed over 36 times the area. Therefore, in situations where the interaction between a stream and an aquifer is of interest, it is important to discretize cell size to approximate river geometry.

(6) Streamflow routing. As discussed previously, the interaction between surface water and groundwater is usually treated as a constant head, constant flow, or as a head-dependent flow boundary. The quantity of surface water in a river, stream, lake, etc., is not accounted for in most groundwater flow simulations. This approach is reasonable for lakes and large rivers where changes in groundwater flow do not appreciably affect the quantity of water in the lakes or rivers. But this approach may not be reasonable for conditions where the amount of surface water is sensitive to changes in groundwater flow. Streamflow routing programs can be used in situations so as not to allow more leakage from streams than there is streamflow. Streamflow routing programs can also allow for the computation of stream stage by inputting an inflow term on the upper reach.

Prudic (1989) developed a simple stream routing computer program, called the "Stream Package," for the U.S. Geological Survey three-dimensional finite-difference flow model MODFLOW. Basic assumptions of the Stream Package are that streamflow entering the modeled area is instantly available to downstream reaches during each time period, leakage between a stream and aquifer is instantaneous, and all stream loss recharges the groundwater system (ET, precipitation, and overland runoff is not accounted for). The Stream Package first computes river stage from Mannings equation (assuming a rectangular channel), then uses the MODFLOW River Package head-dependent flux boundary condition (Equation 6-32) for computing leakage to groundwater flow.

Additionally, the U.S. Geological Survey groundwater flow model MODFLOW has also been coupled to the U.S. Geological Survey unsteady, open-channel flow model BRANCH (Schaffranek et al. 1981). The BRANCH surface flow model simulates flows in networks of open channels by solving the one-dimensional equations of continuity and momentum for river flow. These equations are appropriate for unsteady (changing in time) and nonuniform (changing in location) conditions in the channel. It was developed independent of MODFLOW to simulate flow in rivers without consideration of interaction with the aquifer. BRANCH was modified to function as a module for MODFLOW (Swain and Wexler 1993). Leakance between stream and aquifer is computed through use of a head-dependent flow boundary, Equation 6-31.

c. Modeling interaction between reservoirs (and lakes) and groundwater.

(1) General. Groundwater flow models used to quantify flow between reservoirs (and lakes) and groundwater typically use a specified head to represent the average elevation of the reservoir. However, reservoir levels often show long- and short-term transience in stage and area of inundation. Thus, a model using

specified heads may not provide reliable estimates of groundwater fluxes and reservoir fluctuations over time. As stage increases in reservoirs, a spreading out of the impoundment occurs. Thus, increases in leakage to or from a reservoir are dependent on stage and area of inundation. An algorithm entitled the “Reservoir Package” (Fenske, Leake, and Prudic 1996) was developed for the U.S. Geological Survey three-dimensional finite-difference groundwater flow model MODFLOW to automate the process of specifying head-dependent boundary cells during the simulation. The package eliminates the need to divide the simulation into many stress periods while improving accuracy in simulating changes in groundwater levels resulting from transient reservoir levels. The package is designed for cases where reservoirs are much greater in area than the area represented by individual model cells.

(2) Description. More than one reservoir can be simulated using the Reservoir Package. Figure 6-9 illustrates the specification of the area of potential inundation for two reservoirs. Only those cells specified in the array represented by Figure 6-9 can be activated during model simulations. In cases where areas of higher land-surface elevation separate areas of lower elevations in a reservoir, part of the reservoir may fill before spilling over to an adjacent area. The package can simulate this process by specifying two or more reservoirs in the area of a single reservoir. The area of potential inundation is represented by values of reservoir-bed elevation, layer number, reservoir-bed conductance, and reservoir-bed thickness. Reservoir-bed elevation is the elevation of the land surface within the specified area of potential inundation for each reservoir. Typically, the reservoir-bed elevation at each model cell is equivalent to the average land-surface elevation of the cell.

Reservoir stage is used to determine whether a model cell is activated for each time-step. Whenever stage exceeds land-surface elevation of a cell within the area of potential inundation of a reservoir, the cell is activated. Similarly, whenever reservoir stage is less than the land-surface elevation of a cell, the cell is not activated.

(3) Computation of flow between reservoir and groundwater. Leakage between the reservoir and the underlying aquifer is simulated for each model cell corresponding to the inundated area by multiplying the head difference between the reservoir and the aquifer by the hydraulic conductance. Hydraulic conductance between the reservoir and the aquifer is given by:

$$CRB = \frac{KLW}{M} \tag{6-33}$$

where

CRB = reservoir-bed conductance [L^2/T]

K = vertical hydraulic conductivity of the reservoir bed [L/T]

L = cell length [L]

W = cell width [L]

M = reservoir-bed thickness [L]

Values of reservoir-bed conductance and reservoir-bed thickness can be entered into the Reservoir Package as a single parameter or an array.

Reservoir-bed thickness is subtracted from reservoir-bed (or land-surface) elevation to obtain the elevation of the reservoir-bed bottom. The reservoir-bed bottom elevation is used in computing leakage. When the head in the aquifer is above the reservoir-bed bottom, leakage from or to the aquifer is computed by:

$$Q_{res} = CRB(h_{res} - h_{gw}) \tag{6-34}$$

where

Q_{res} = leakage from the reservoir [L^3/T]

h_{res} = head in the reservoir [L]

h_{gw} = aquifer head [L]

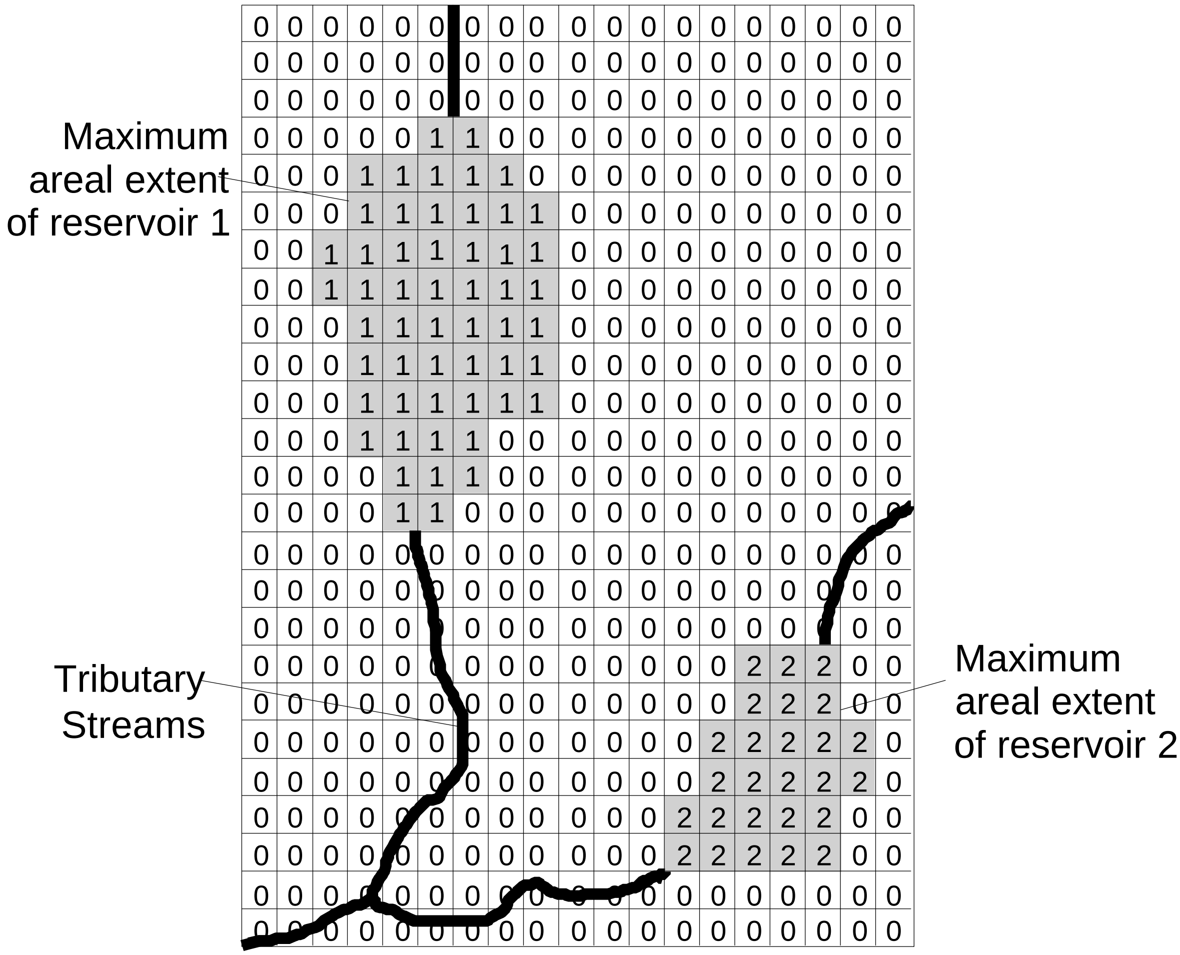

Figure 6-9. Definition of maximum areal extent of reservoir(s)

When the head in the aquifer is less than the elevation of the reservoir-bed bottom, leakage from the reservoir to the groundwater is computed by:

$$Q_{res} = CRB(h_{res} - h_{resbot}) \qquad (6\text{-}35)$$

where

h_{resbot} = elevation of the reservoir-bed bottom [L]

(4) Reservoir package applicability and limitations. Water exchange between surface and subsurface is instantaneous, and it is assumed that there is no significant head loss between the bottom of the reservoir bed and the water table. Water exchange takes place across the horizontal faces of model cells. Thus, bank flow is not directly simulated. The effects of bank flow can be approximated by dividing the reservoir into multiple layers. Changes in reservoir stage are transmitted instantly across the reservoir. Implied in this assumption is that the reservoir has no slope and there is no flow across the reservoir. This assumption may not be valid for large reservoirs. Additionally, head-dependent flow boundaries are

specified for all cells having a land-surface elevation less than the reservoir stage, even if areas of higher land-surface elevations separate areas of lower elevations. This assumption may be unreasonable for reservoirs in which the land surface is uneven and where parts of the reservoir fill before spilling into adjacent lower-lying areas. The package can simulate this process by having two or more reservoirs specified. Neither precipitation on nor evapotranspiration from the reservoir is directly simulated; however, both can be included by adding or subtracting an equivalent volume of water per unit area from the flood stage.